AMERICAN CONCRETE INSTITUTE

PROCEEDINGS

OF THE

TWELFTH ANNUAL CONVENTION

Held at Chicago, Ill.,

February 14, 15, 16, 17, 1916

VOLUME XII

EDITED BY THE SECRETARY
AND
JOHN M. GOODELL

PUBLISHED BY THE INSTITUTE
1916

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	1906–9	Merrill Watson.
	1909–11–12	Edward D. Boyer.
	1913–14	A. N. Talbot.
	1915	William K. Hatt.
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	1911–12	A. N. Talbot.
	1913–14	L. C. Wason.
	1915	Henry C. Turner.
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	1908	S. B. Newberry.
	1909–12	E. S. Larned.
Fourth Vice-President.	1905–7	A. Monsted.
	1908–9	George C. Walters.
	1909–10	F. A. Norris.
	1911–12	Ira H. Woolsen.
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	1906–12	H. C. Turner.
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	1907	W. W. Curtis.
	1908–9	George C. Wright.
	1910	Edward E. Krauss (Acting).
	1911–14	Edward E. Krauss.
	1915	Charles L. Fish.
	1915	J. M. Goodell (Acting).

Directors.	1913	BENJAMIN F. AFFLECK. WILLIAM P. ANDERSON. EDWARD D. BOYER. W. L. CHURCH. CHARLES DERLETH, JR. ERNEST L. RANSOME.
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	1915	WILLIAM P. ANDERSON. ERNEST ASHTON. EDWARD D. BOYER. WILLIAM H. HAM JOHN B. LEONARD. ALFRED E. LINDAU.

BY-LAWS.

Article I.

MEMBERS.

Section 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

Sec. 2. A firm or company shall be treated as a single member.

Sec. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

Sec. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at that time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

Sec. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

Article II.

OFFICERS.

Section 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Direction.

Sec. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

Sec. 3. There shall be a Committee of five members on Nomination of Officers, elected by letter ballot of the members of the Institute, which is to be canvassed by the Board of Direction on or before September 1 of each year.

The Committee on Nomination of Officers shall select by letter ballot of its members, candidates for the various offices to become vacant at the next

Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 60 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 20 days thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 30 days before the Annual Convention to the members of the Institute for letter ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

SEC. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-Presidents and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty, of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

SEC. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election, appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

SEC. 7. The Board of Direction shall appoint a Committee on Resolutions, to be announced by the President at the first regular session of the annual convention.

SEC. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

SEC. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

SEC. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

SEC. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

SEC. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

SEC. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

ARTICLE III.

MEETINGS.

SECTION 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of the Convention.

SEC. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

SEC. 3.—The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

SEC. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

ARTICLE IV.

DUES.

SECTION 1. The fiscal year shall commence on the first of July and all dues shall be payable in advance.

SEC. 2. The annual dues of each member shall be ten dollars ($10.00).

SEC. 3. Any person elected after six months of any fiscal year shall have expired, need pay only one-half of the amount of dues for that fiscal year; but he shall not be entitled to a copy of the Proceedings of that year.

SEC. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon the payment of all indebtedness against them upon the books of the Institute.

Article V.

RECOMMENDED PRACTICE AND SPECIFICATIONS.

Section 1. Proposed Recommended Practice and Specifications to be submitted to the Institute must be mailed to the members at least thirty days prior to the Annual Convention, and as there amended and approved, passed to letter ballot, which shall be canvassed within sixty days thereafter, such Recommended Practice and Specifications shall be considered adopted unless at least 10 per cent of the total membership shall vote in the negative.

Article VI.

AMENDMENT.

Section 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF THE PROCEEDINGS OF THE TWELFTH ANNUAL CONVENTION.

First Session, Monday, February 14, 1916, 10 a. m.

The Convention was called to order by the President, Leonard C. Wason.

The President announced the appointment of the following Committee on Resolutions:

A. J. Maynard, *Chairman*, State Farm, Mass.
William P. Anderson, Cincinnati, Ohio.
William K. Hatt, Lafayette, Ind.
Richard L. Humphrey, Philadelphia, Pa.
E. J. Mehren, New York, N. Y.

The following papers were then read and discussed:

"The Use of Concrete at the State Farm at Bridgewater, Mass.," by A. J. Maynard and B. Baker.

"Relining a Tunnel with Steam-Jetted Concrete," by Harold P. Brown.

"Some Features of Concrete Work on Subway Construction, New York City," by Robert Ridgway; read by E. J. Mehren in the absence of the author.

The report of the Committee on Nomenclature was, in the absence of the Chairman, Frank C. Wright, presented by L. R. Ferguson.

On motion it was decided that the adoption of the definitions proposed by the Committee be deferred and the suggestion of the Committee be adopted that there be organized a joint conference to consist of representatives from various engineering societies to consider such definitions.

On motion it was decided to reconsider the report of the Committee at the session on Wednesday afternoon.

The meeting then adjourned until 8 p. m.

Second Session, Monday, February 14, 1916, 8 p. m.

President Leonard C. Wason in the chair.

The report of the Committee on Insurance was presented by H. C. Turner in the absence of the Chairman, J. P. H. Perry.

In the absence of the author, the paper on "Unit Cost in Construction," by Sanford E. Thompson, was read by the Secretary.

Robert F. Havlick, Chairman, presented the Report of the Committee on Building Blocks and Cement Products.

The report of the Committee was accepted and ordered printed so that the proposed specifications could be adopted at the next convention.

The meeting then adjourned until Tuesday at 10 A. M.

THIRD SESSION, TUESDAY, FEBRUARY 15, 1916, 10 A. M.

President Leonard C. Wason in the chair.

E. J. Moore, Chairman, presented the report of the Committee on Reinforced Concrete and Building Laws, which was discussed and referred back to the Committee for further consideration.

The following papers were then read and discussed:

"Some Suggestions for the Design of Concrete Buildings," by W. P. Anderson.

"Reinforced Concrete Columns," by P. P. Furber.

The meeting then adjourned until 8 P. M.

FOURTH SESSION, TUESDAY, FEBRUARY 15, 1916, 8 P. M.

President Leonard C. Wason in the chair.

William K. Hatt presented a paper on "Genesis of Reinforced Concrete Construction."

The following papers were then read and discussed:

"A Further Discussion of Steel Stresses in Flat Slab Floors," by H. T. Eddy; in the absence of the author read by N. H. Johnson.

"Design of Reinforced Concrete Footings for Buildings," by R. L. Bertin; in the absence of the author read by Frank C. Wight.

The report of the Committee on Fireproofing, John Stephen Sewell, Chairman, was presented by the Secretary.

The report of the Committee on the Treatment of Concrete Surfaces was presented by the Chairman, Cloyd M. Chapman, and after discussion was approved.

The meeting then adjourned until Wednesday at 10 A. M.

FIFTH SESSION, WEDNESDAY, FEBRUARY 16, 1916, 10 A. M.

President Leonard C. Wason in the chair.

The following papers were read and discussed:

"Time Tests of Concrete," by Almon H. Fuller and Charles C. More; read by Professor MacMillan.

"The Flow of Concrete Under Sustained Load," by Earl B. Smith.

"Influence of Temperature on the Strength of Concrete," by A. B. McDaniel.

"Tests of Concrete Columns, Plain and Reinforced," by Frank P. McKibben and A. S. Merrill; presented by E. J. Moore in the absence of the authors.

"Possibilities of Unit Concrete and Structural Steel as a Means of Meeting the Speed and Engineering Requirements of Modern Building Construction," by Charles D. Watson.

The reports of the following committees were then presented:

Committee on Reinforced Concrete Highway Bridges and Culverts, T. McDonald, Chairman.

Committee on Sidewalks and Floors, L. R. Ferguson, Chairman.

Business Session.

The President presented the Annual Report of the Board of Direction.

The Treasurer, Robert W. Lesley, presented his annual report which was accepted and ordered filed.

The President reported that the letter ballot had resulted in the election of the following officers for the ensuing year:

President: Leonard C. Wason.
Vice-President: Henry C. Turner.
Treasurer: Robert W. Lesley.
Directors· *First District*—Charles R. Gow.
Second District—Edward D. Boyer.
Sixth District—John G. Treanor.

On motion a unanimous vote of thanks was extended to Mr. John M. Goodell in appreciation of his earnest and valuable services as Acting Secretary during the past six months.

The meeting then adjourned until 2 P. M.

SIXTH SESSION, WEDNESDAY, FEBRUARY 16, 1916, 2 P. M.

President Leonard C. Wason in the chair.

The Committee on Nomenclature, F. C. Wight, Chairman, presented its revised report and the question of submitting the proposed definitions to letter ballot was ordered left to the discretion of the Board of Direction.

The following papers were then read and discussed:

"Construction of Reinforced Concrete Factory Building with Submerged Foundations under Severe Tidal Conditions," by N. M. Loney; in the absence of the author presented by E. J. Moore.
"Forms for Concrete Work," by R. A. Sherwin.
"The Proper Use of Concrete Gravity Chutes," by W. H. Insley and C. C. Brown.
"The Concrete Viaducts and Bridges of Cincinnati," by Frank L. Raschig.
"Construction Methods on the Tunkhannock and Martin's Creek Viaduct, Lackawanna Railroad," by C. W. Simpson, in the absence of the author read by title.
"Tests of Large Reinforced Concrete Slabs," by A. T. Goldbeek and E. B. Smith.

The Report of the Committee on Reinforced Concrete Chimneys, H. W. Latta, Chairman, was presented by L. R. Ferguson.

The meeting then adjourned until 7 P. M.

SEVENTH SESSION, WEDNESDAY, FEBRUARY 16, 1916, 7 P. M.

President Leonard C. Wason in the chair.

The following papers were presented and discussed:

"Motion Picture Studies on the Making and Placing of Concrete," by Nathan C. Johnson.
"The Construction of the Toronto Hamilton Highway by Day Labor," by H. S. Van Scoyoc.

The Report of the Committee on Concrete Roads and Pavements was then presented by the Chairman, Arthur N. Johnson, and accepted.

The paper on "Concrete Foundations for Asphalt Pavements and Roads Subject to Heavy Travel," by Clifford Richardson, was in the absence of the author read by title.

The following papers were then read and discussed:

"Foundations for Permanent Pavements," by R. C. Stubbs.
"The Construction of the Easton-Allentown Road," by John T. Gephart, Jr.; in the absence of the author presented by L. R. Ferguson.
"The Coleman Du Pont Road, Delaware," by Charles Upham; read by S. F. Butler.
"Essential Features for Successful Construction of Concrete Roads," by William M. Acheson; in the absence of the author the paper was read by title.

The meeting adjourned until Thursday at 10 A. M.

EIGHTH SESSION, THURSDAY, FEBRUARY 17, 1916, 10 A. M.

President Leonard C. Wason in the chair.

George A. Rankin presented a paper on the "Chemistry of Portland Cement."

The report of the Committee on Concrete Aggregates, Sanford E. Thompson, Chairman, was presented by Cloyd M. Chapman, and discussed.

Mr. Cloyd M. Chapman presented a paper on "The Use of the Universal Sand Tester."

The meeting then adjourned until 2 P. M.

NINTH SESSION, THURSDAY, FEBRUARY 17, 1916, 2 P. M.

President Leonard C. Wason in the chair.

The following papers were read and discussed:

"Reinforced Concrete in Sewer Construction," by W. W. Horner.
"The Fall River Concrete Conduits," by Frederic H. Fay.
"Durability of Concrete Pipe," by J. H. Libberton.
"Construction of Kensico Dam," by Wilson F. Smith; read by title.
"Construction of the Austin, Texas, Reservoir and Dam," by L. Lyndon and F. S. Taylor; read by Mr. Hayn.

The Committee on Resolutions presented the following report which was unanimously adopted:

Resolved, That the thanks of the American Concrete Institute be tendered to the men who kindly contributed papers to the Convention.

Resolved, That the President be hereby instructed to send a letter of thanks on behalf of the Institute to each person contributing a paper.

A paper on "The Middleborough, Mass., Reinforced Concrete Water Tower Tank" was presented by G. A. Sampson.

The President then declared the convention adjourned *sine die*.

American Concrete Institute

PROCEEDINGS

OF THE

TWELFTH CONVENTION

This Institute is not responsible, as a body, for the statements and opinions advanced in its publications.

GENESIS OF REINFORCED CONCRETE CONSTRUCTION.

By W. K. Hatt.*

Prefatory Notes.

It is the custom in some institutions of learning that some one of the officials shall give, each year, an address entitled "In Praise of the Founders." It has occurred to the present speaker that it would be appropriate on this occasion to discuss, in a very incomplete manner it must be confessed, the contributions of the early workers in the field of reinforced concrete.

This service could only be completely rendered after a long and difficult research. Examples of constructions are given in books without the date of their design. The history of these early periods is scattered through sources that cannot be attained in any one library.

The speaker has consulted all the resources at his disposal in the three languages of French, German and English, endeavoring to go back in every case to the original sources. Where this was not possible the excellent treatise of Christophe has been relied upon. He has endeavored to avoid any questions of a controversial nature.

If this address will impress upon our present-day engineers the extent of the achievements of the early builders and investigators and will stir up an interest on the part of someone to carry forward an historical account of the development of reinforced concrete, its purpose will have been fulfilled.

Early Ideas in Reinforced Concrete.

As in the history of other arts, we may find early disclosures of the principles of action of various later constructions.

* Professor of Civil Engineering, Purdue University, Lafayette, Ind.

The early date at which reinforcement was considered is instanced in the following quotation from *Bridge Engineering*, by H. G. Tyrell, 1911, page 407, Chapter 17, on "Reinforced Concrete Bridges."

"In the construction of masonry arches, it has long been observed that the arches settle at the crown when the temporary centers are removed, and the extrados joints tend to open at the haunches, these points being known as points of rupture. To prevent these joints from opening, rods and iron bands have often been used in the extrados of the arch rings extending from the piers and abutments up to or beyond the points of rupture. Brunel, when experimenting with arches, built a semi-arch of brick 60 ft. long with hoop iron bond which supported itself by cantilever action." (About 1835.)

"Sir Shafto Adair built a concrete bridge in 1871 over Waveney at Homersfield, England, with model reinforcing frames from designs by H. M.

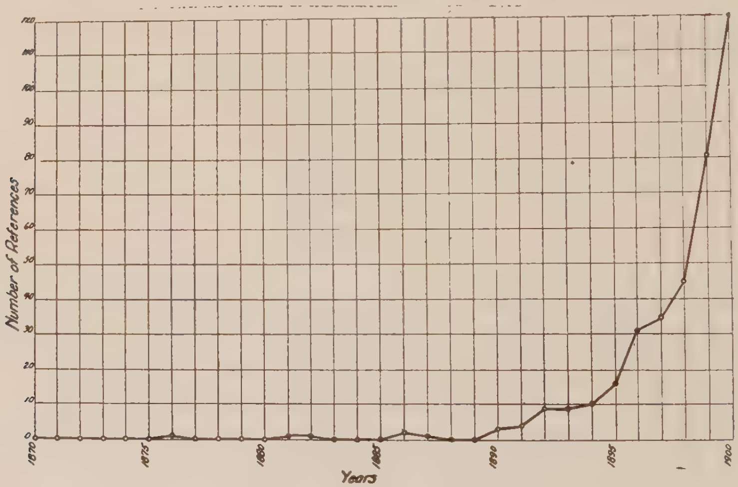


FIG. 1.—CHART SHOWING NUMBER OF REFERENCES.

Eyton of Ipswich. The arch had a span of 50 ft., a rise of 5 ft. 3 in. and the skeleton iron frames were imbedded in Portland cement concrete, over 100 tons of concrete being used."

We know that as early as 1838 the action of iron ties to give flexural strength to brick masonry was well understood. The two materials were shown to act as one composite material. [See Proc. Inst. C. E., Vol. 1, 1838, pp. 16 and 20.] It would seem to be a natural step to use such iron ties in concrete.

Many will be surprised to learn of the extent of the remarkable development of reinforced concrete construction in Europe during a period when the art was in a crude state in this country. In Europe, cheap labor and high cost of materials, together with a reliance upon theory and the certainty of good workmanship led to constricted sections. There the art was developed

by commercial companies and had little publication of actual designs, or little recognition by officials during its early rapid growth.

To determine a date back of which this literature might be examined is somewhat an arbitrary selection. I, however, have consulted the very complete bibliography found in Chapter 10 of the French treatise by R. Feret, entitled "Étude Expérimentale du Ciment Armé." The references in this bibliography cover the principal countries of Europe and the United States, and apparently no publication of even slight importance has escaped. Plotting the number of references of succeeding years, as shown in Fig. 1, it is apparent that the years prior to 1900 may fairly be considered as early.

The French Exposition at Paris in the year 1900 contained such advance applications of reinforced concrete as to mark an epoch. Just before this date, we find appearing in the *Annales des Travaux Publique de Belgique* a series of articles by P. Christophe, afterwards published in an enlarged form in a well-known volume by Christophe entitled "Le Beton Arme." This series of articles was marked by the publication of large plates of drawings showing the actual reinforcement of various structures, with dimensions. Previous to this time there had been photographs of structures and skeleton drawings, but it would appear that the actual designs were considered to be trade secrets.

For these three reasons, namely, the display of the work at the French Exposition, the appearance of the Christophe articles, and the state of the literature as shown by the bibliography, it seemed to the writer that he might most profitably discuss the literature mainly prior to 1895.

LITERATURE PRIOR TO 1900.

The writer must confess that this paper cannot be complete, because certain of the early publications have not come to his hands in their entire form, especially the publication "Le Ciment" and "Le Beton Armé."

The original book of Coignet was published in 1861. In this he has described the advantages of the use of iron in masonry or in concrete. In construction of floors, Coignet has indicated the method of construction in iron and concrete as follows:

"Below this network of iron, a wooden falsework is placed; on this falsework is turned the concrete in thin successive layers and is tamped vigorously. It raises itself little by little up to the beams and envelopes them entirely and finally covers them with a layer of from 5 to 6 centimeters.

"At the end of some days, the concrete having attained the hardness of rock, the falsework is taken away and there remains a true slab of concrete forming a ceiling underneath and a floor above.

"In this system of flooring the iron work is completely enclosed in a slab of hard rock. One sees then that an iron work so placed in stone cannot be bent without bending the stone itself."

The increase of resistance to tension which comes to the cement by the introduction of iron is brought to view very clearly in the following paragraph:

"Concrete offers great resources—not only over and above a mass of

ordinary masonry, but it possesses a resistance to tearing apart of 10 kg. per sq. cm. It is also possible to introduce, during its manufacture at the time of tamping, ties or iron chains of such a kind that by this means the strength of the concrete, already so large, is increased by these materials that are introduced."

Coignet's patent of 1869 is for improvements in artificial stone monolithic structures. It "consists in the introduction in the body of the structure of double-headed nails, double T-pieces, clamps, hoops, scraps of twisted or irregular shaped irons for the purpose of strengthening same and giving it greater cohesive strength. The irons to be thus introduced may be arranged in such a manner as to interlace each other, so that by the combination of this metallic skeleton, and of agglomerated artificial-stone paste, the thickness of the walls or size of articles may be considerably reduced and yet great strength be attained."

He mentions pipes, troughs or waterways. In the latter, "angular bent angle or L-shaped pieces may with good effect be introduced in the body of the material, to give greater strength to the angles and prevent the trough from spreading asunder at those points."

He claims no novelty in use of iron clamps or framework of metal in ordinary masonry or brickwork for strengthening same.

His claims are for:

(1) The combination of agglomerated artificial stone paste with iron straps of irregular shape, such as nails, double-headed nails or bolts, rings, hooks, clamps, wire, substantially in the manner and for the purpose set forth.

(2) The introduction in the body of artificial stones, or in the body of artificial stone monolithic structures made of agglomerated artificial stone paste, of skeletons or metallic framework linked or arranged so as to strengthen the same, substantially as specified.

(3) The application of agglomerated artificial stone paste to the protection and isolating of telegraphic wires.

Monier's descriptive memoirs of August, 1873, with drawings, shows his system of reinforced concrete arch rings. Round rods are used, constituting an intersecting grillage which is in one, two or three layers, depending upon the span and load. These are thus round bars in extrados and intrados not connected with metal through the thickness of the arch ring.

Monier's Patent Specification of September 15, 1890, Patent No. 208,250, applies to cement constructions with iron skeleton adapted for reservoirs, silos, tanks, cisterns, etc. In erection of reservoirs, etc., he prepares the foundation frame, and then puts in the first heavy irons necessary to give them their shape and desired dimensions as well as great resistance, as shown in his drawings. He describes in detail the method of constructing a reservoir resting on poor soil and having a diameter of more than 20 m. He describes the use of concrete columns with spiral and longitudinal reinforcements, and counterforts to reservoir walls or pipes of similar construction. His concrete is of mortar, of lime or cement, with sand or gravel slag.

In addition to the work of Coignet and Monier, we may especially mention certain early publications. The book printed by Thaddeus Hyatt

(Fig. 2) in 1877 for private circulation, whose title page is shown in Fig. 3, discloses the improvements in fireproof construction, describing certain tests of reinforced concrete beams and the principles of action of these.

According to an article by Professor Spofford, Hyatt was a lawyer by education, born in New Jersey in 1816, and lived most of his life in New York and London. He died in 1901.

Hyatt compares the first successful application of cast iron beams in buildings in 1801 with the method of construction shown in Fairburn's book

FIG. 2.—PORTRAIT OF THADDEUS HYATT.

of 1854, and asserts that the only difference between the early 1801 and later type of structure, 1877, was the form of the beam.

Hodgekinson's experiments in 1827 30 on metal beams had shown the economy of the I-over the T-form. In 1849–50, through the experiments to determine the construction of Stephenson's tubular bridges, wrought iron was substituted for cast-iron beams.

He calls attention to the fact that iron was believed to be, but is not,

a fireproof material; that the bottom of the iron beams, against which the floor arches abut, will soften by heat, and the construction will collapse. The system adopted by Hyatt proceeds on the assumption that iron is not fireproof, and that in a floor it must be protected with a fireproof coating on all sides, the amount of which he has determined from experiment. His constructions show girders protected on all sides with concrete and light iron joists carrying a slab between the girders. And between these joists run vertical bars connected by wires to hold the ceiling.

He shows a construction in which floor joists are dispensed with, "flat tie-irons being substituted for the joists, the concrete in this case becoming the compressive member of the beam or slab."

He shows why it is a waste of material to use iron in beam form in concrete. His diagram shows a slab first with I-beams embedded, and second with only the lower flange of the I-beam left, a section of 2 x $\frac{1}{4}$ in. spaced 12 in. c.c. in a slab of 10 in. depth. Hyatt takes the axis at half the depth of the beam, as did Koenen in 1887. He then has 60 sq. in. of concrete in compression as the fulcrum for the tie to act against in lieu of the $\frac{1}{2}$ in. of metal in the top flange of the rolled joists. The question is, can those 60 in. of concrete when brought into compression be made to do the duty of the $\frac{1}{2}$ in. of metal at the top of an iron joist? He computes the compressive resistance of the concrete at 2000 lb. per sq. in. and has a mean of 1000 lb. acting at $2\frac{1}{2}$ in. above the neutral axis, or $60 \times 1000 \times 2\frac{1}{2} = 150{,}000$ in. lb. moment in compression. The moment of the metal in tension is $60{,}000 \times 2 \times \frac{1}{2} \times 4 = 120{,}000$ in. lb.

He therefore concludes that the compressive surface of the concrete is in excess of the demands of the tie metal.

Then he refers to his experiments to show the possibility of uniting metal to concrete as a bottom flange is held to its web in a rolled or riveted beam and that the two metals will act in concert. His experiments include beams as follows:

One of cement, one of concrete, four with firebrick tiles, and the remainder of concrete with various forms of reinforcement, round rods, iron plates of various width placed both vertically and flatwise in the beam. In certain beams some of the round rods are turned up from the bottom to the top of the beam. Rods ending in the top of the beam have heel plates. The plate ties when placed vertical are connected by rods running transversely in the beam. Plate ties placed horizontally are anchored to the top of the beam by bolts.

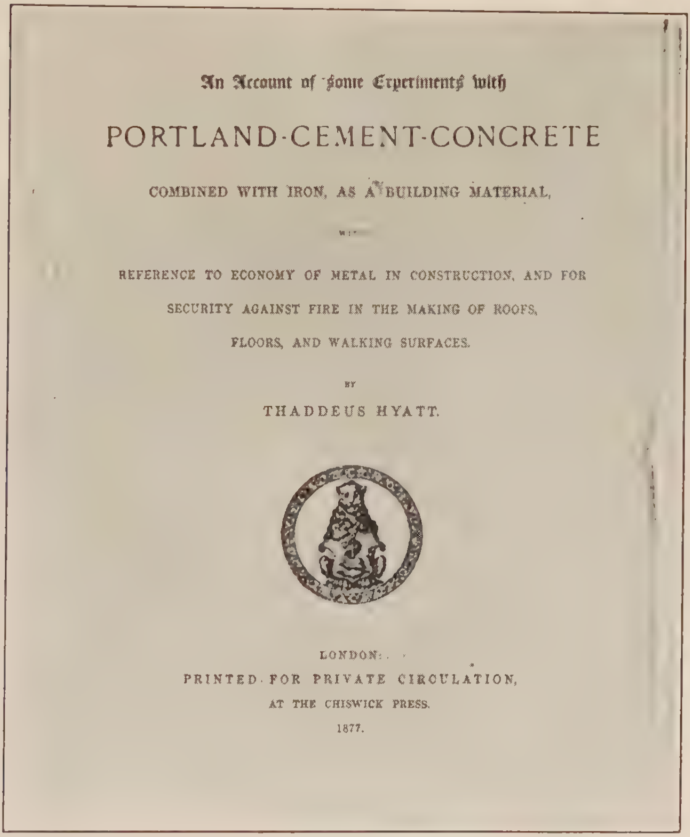
The beams varied in cross-section, 8 x 12, 9 x 8, 12 x 12 in., and were tested on a 5-ft. or an 8-ft. span. Of course, we now know that a 12-in. beam on a 5-ft. span will give rise to large shearing and bond stresses. His reinforcement was from 1.37 per cent.

His tests proved to him that the two materials worked in harmony.

Hyatt had evidently an analytical faculty. He sees that other matters besides strength need investigation. The fireproof qualities of Portland cement; the ratio of its expansion and contraction as compared with iron under like conditions; the effect of the two in combination when heated;

the heat-conducting powers of concrete; laws of its conduction; the compressibility and extensibility of cement as compared with iron.

Hé tests fireproof qualities and conduction. He found the expansion of concrete to be 0.00137 for 180° F., as compared with 0.00140 for wrought iron, and he finds also that no differential action exists in actual reinforced



An Account of Some Experiments with

PORTLAND-CEMENT-CONCRETE

COMBINED WITH IRON, AS A BUILDING MATERIAL,

WITH

REFERENCE TO ECONOMY OF METAL IN CONSTRUCTION, AND FOR SECURITY AGAINST FIRE IN THE MAKING OF ROOFS, FLOORS, AND WALKING SURFACES.

BY

THADDEUS HYATT.

LONDON:
PRINTED FOR PRIVATE CIRCULATION,
AT THE CHISWICK PRESS.
1877.

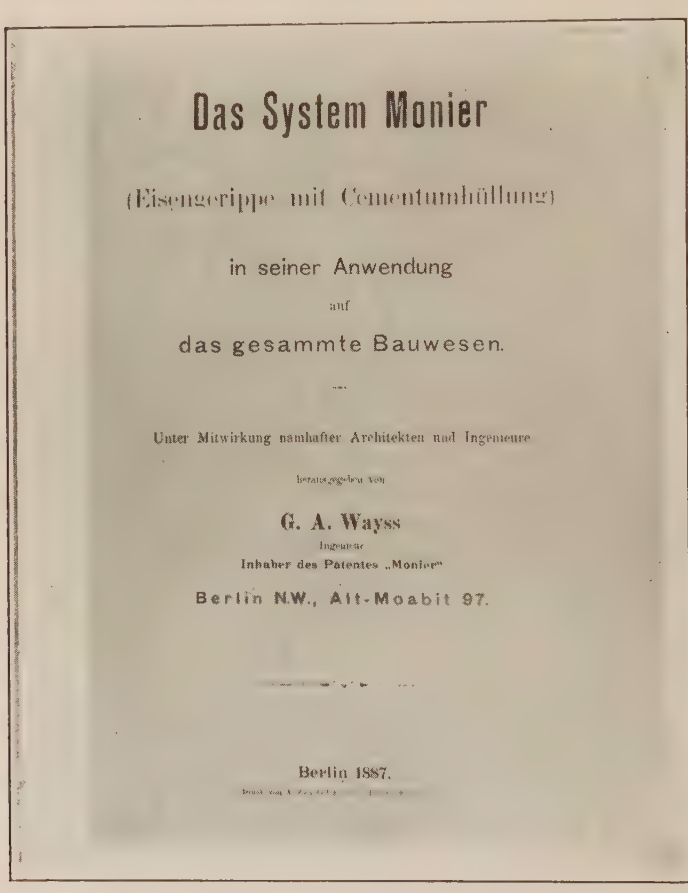
FIG. 3.—TITLE PAGE OF HYATT'S BOOK.

concrete bars when heated in the fire. His tests for the modulus of elasticity in compression show 0.00096 for 1000 lb. per sq. in., or $E = 1{,}000{,}000$ lb. per sq. in. approximately.

In tension the extension was 42/1,000,000 part of its length at a breaking stress of 100 lb. per sq. in. Here $E = 2{,}240{,}000$. Hyatt reasons that it was not possible to obtain the tensile modulus with the same accuracy as

the compressive modulus. In addition to his laboratory tests, he applies the test of fire and water and a load of 200 lb. per sq. ft. to a real floor section 6 ft. long by 2 ft. wide by $7\frac{1}{2}$ in. thick, containing tie metal in the gridiron form in the middle of its thickness.

The fire was incandescent, bed 6 in. thick, 12 in. under the slab, kept



Das System Monier

(Eisengerippe mit Cementumhüllung)

in seiner Anwendung

auf

das gesammte Bauwesen.

Unter Mitwirkung namhafter Architekten und Ingenieure

herausgegeben von

G. A. Wayss

Ingenieur

Inhaber des Patentes „Monier“

Berlin N.W., Alt-Moabit 97.

Berlin 1887.

[illegible]

FIG. 4.—TITLE PAGE OF DAS SYSTEM MONIER.

for 10 hours, during which the deflection increased from $\frac{1}{32}$ in. to $\frac{3}{8}$ in. Then a stream of cold water was thrown on the bottom of the slab for 15 to 20 minutes. The under side of the slab was found to be unimpaired and when cold had returned to its original shape without deflection. A second test was made on the same slab. He also experimented with plaster of

Paris, concretes more or less porous and also with air spaces, and found that the best material to protect the metal against heat was that which was strongest in compression, viz, Portland cement concrete of the best quality, no advantage for any purpose being found from fibers of any kind, not even asbestos.

On speaking of certain of his test beams in which flat ties were connected by bolts with the upper part of the beam, he says that no contrivance of this kind is necessary. The tie requires no attachment to it other than one which will prevent it from sliding upon the web. Hyatt's analyses shows him to be in possession of the mechanics of beam action. He prefers the flat tie on account of its greater holding surface. We know now that

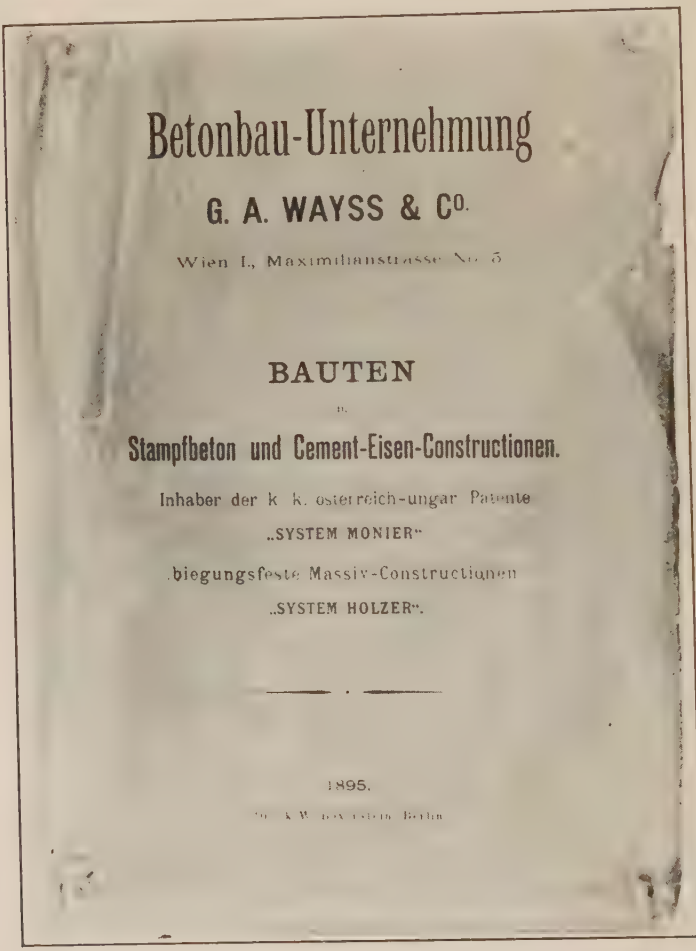
FIG. 5.—TITLE PAGE OF REHBEIN.

this flat tie is not efficient in bond. He puts the flat tie edgewise in the beam and connects the ties with horizontal transverse wires that serve to prevent sliding. He states that hollow floors may be made by using the ties for the top flange as well as for the bottom flange. Floors of 40 ft. span he says need to be 2 ft. thick, with bars 2 x $\frac{1}{4}$ in. spaced 2 in. apart for a load of 223 lb. per sq. ft.

Hyatt's "stone light" was well known, a combination of his gridiron slabs with glass inserts. He says he is prepared to undertake the construction of domes of any span from 2 ft. to 200 ft. Arch roofs with lights are recommended for picture galleries. He outlines the possibility of the use of his construction for chimneys and lighthouses, stairways, bridges, etc.

He acknowledges his debt to Mr. Thomas Beckett of Birmingham, an engineer, for assistance in working out his problems.

In this book of date November, 1877, we have the first recorded broad and definite treatment of reinforced concrete, and an attempt to establish an experimental basis for design.



Betonbau-Unternehmung

G. A. WAYSS & Co.

Wien I., Maximilianstrasse No. 5

BAUTEN

in

Stampfbeton und Cement-Eisen-Constructionen.

Inhaber der k. k. österreich-ungar. Patente

„SYSTEM MONIER"

biegungsfeste Massiv-Constructionen

„SYSTEM HOLZER".

1895.

FIG. 6.—TITLE PAGE OF BETONBAU-UNTERNEHMUNG.

The publication in 1877, entitled *Das System Monier,* describes the advantages of reinforced concrete, makes the record of certain tests and explains the method of calculations derived from the tests of the firm of Wayss and by Engineer Koenen. The title page of the book is shown in Fig. 4.

A further description of the work of the firm of Wayss & Co. is found in the publication by Rehbein in 1894 entitled "Monier und Beton Bauwerke." A number of interesting bridges are shown in this publication (Fig. 5).

The publication entitled "Betonbau-Unternehmung" of Wayss, in 1895, shows some excellent photographs of the structures to that date (Fig. 6).

Certain publications describing the doings of special companies, for instance in France, *Le Ciment*, appearing monthly after 1896. In Germany a journal, *Tonindustrie Zeitung*, contained many articles on cement and concrete. The publication, *Zement und Beton*, from 1902 is especially devoted to the concrete industry. In France, *Le Béton Armé* was the organ of the agents of the Hennebique system and appeared monthly after June, 1898. A very important journal, *Beton und Eisen*, published at Vienna by von Emperger, was created in 1902, and published monthly after 1905. This may be considered an international journal on reinforced concrete.

The book by P. Christophe, entitled "Béton Armé," is probably the best complete work on reinforced concrete from the descriptive and the historical side. He is not content with illustrating the works, but has given dates of the appearance of a large part of the constructions of our illustrations. The book is not only valuable matter, but is written in excellent style.

The work of Berger and Guillerme, published in 1902 by Dunod, Paris, is especially valuable in the description of the various systems of construction with an account of the works and methods of calculations by these systems. It is weak in theoretical treatment and account of experiments. Curiously enough, in a work which has for its main field historical records, the authors have in very few cases given dates.

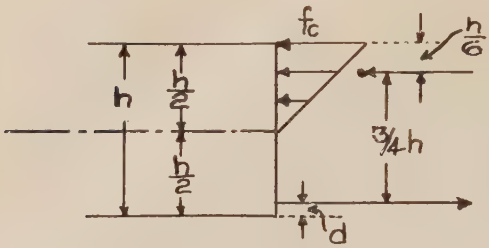
A very excellent work from the standpoint of discussion of experiments and theories and regulations is the book by Professor E. Mörsch of Zurich, entitled "Concrete-Steel Construction." This has been translated by Mr. E. P. Goodrich and published by the *Engineering News* Publishing Co. in 1910. The work was originally published by Wayss and Freytag in 1902 and then re-published by Mörsch in 1905.

A work in the English language from the press of Van Nostrand under the authorship of Marsh is a clear and readable account of the European work as disclosed in the books by Christophe and Berger.

METHODS OF CALCULATION.

Methods of calculating the strength of these structures were defined by the promoters of the various systems of construction as early as 1876.

The various Monier constructions built throughout Germany and Austria by the firm of Wayss & Company were based upon the method of design developed for this firm by Koenen. Koenen assumed a rectilinear variation of compressive stress; the tensile stresses were neglected. His equations are as follows:



b = width of beam

In practice $d = \frac{h}{12}$

$$\frac{f_c h}{4} = f_s A_s$$

$$M = \frac{f_c h}{4} b \frac{3}{4} h = \frac{3}{16} f_c b h^2$$

$$A_s = \frac{1}{4} \frac{f_c}{f_s} hb \text{ and}$$

$$h = 2.31 \sqrt{\frac{M}{f_c b}}$$

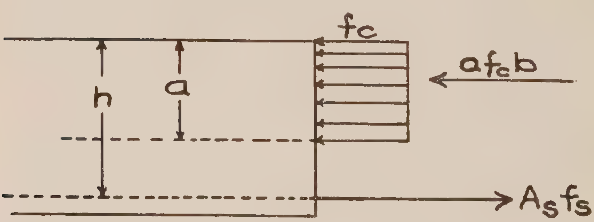
For fixed ends $M = \frac{1}{24\, w\, l^2}$ at center.

Christophe says that the error in locating the neutral axis at one-half the depth has this effect, that for 1 per cent reinforcement the compressive stresses are really about 50 per cent greater than Koenen fixed them, and the tensile steel stress is but 8 per cent less.

The method used by Hennebique is evidently very faulty; he assumed the compressive stress to be uniform above the neutral axis. He omits the tensile stress on the concrete. He supposed that the moment of the compressive stresses equaled the moment of the force of the steel. That is to say that either one of these equaled one-half of the external moment. Actually, as we now know, the concrete supplies about one-fourth and the steel and concrete in tension altogether three-quarters of the resisting moment. In working calculations, the height of the beam was supposed to be known. Hennebique's equations follow below.

$$\text{Moment of compression} = \frac{f_c a^2 b}{2} \quad \text{.................. (1)}$$

$$\text{Moment of tension} = A_s f_s (h-a) \quad \text{.................. (2)}$$



(1) = (2) and one-half of the external moment equals either (1) or (2). When b and h are known, h and A are calculated.

Generally h is assumed and the other quantities calculated. Hennebique used 25 kg. per sq. cm. for the concrete and 10 kg. per sq. mm. for iron.

In construction $h = \frac{3}{2}a$, for small h, to $h = 5/2a$ for large h.

Christophe shows that Hennebique's formulas yield a factor of safety that depends upon the height of the piece. For shallow slabs where Hennebique calculates 25 kg. per sq. cm. in compression and 10 kg. per sq. mm. in tension, he really has 60 kg. and 7.8 kg. respectively. For thick slabs he really has 38 kg. and 12.86 kg. respectively.

For fixed beams $M = \frac{1}{10} WL$. For two-way slabs $M = \frac{1}{36} WL$.

Melan, in 1896, proposed a method involving a fictitious cross-section in which the dimensions of steel reinforcing were increased in proportion to the ratio between coefficients of elasticity of steel and concrete. Indeed, this method was clearly shown by the late J. B. Johnson in the *Engineering News* of January, 1895.

The recognition of the different properties of the two materials and the location of the neutral axis appeared as early as 1894, when the paper by Coignet and Tedesco, published in *Proceedings* of the Soc. des Ingénieurs Civils, approximated nearly all modern theories of flexural strength. The differing moduli of the two materials were taken account of and a quadratic equation fixed the location of the neutral axis.

An article in *Le Ciment*, in 1896, refers to P. Planat as one of the first writers upon the theory of reinforced concrete. P. Planat was the editor of the journal of architectural construction entitled *Le Construction Moderne*. In the journal after December, 1893, Planat published a series of articles of the theory of reinforced concrete which afterwards appeared in book form without date under the title of "Recherches sur la Théorie des Ciments Armés," Aulanier & Co., Paris. Planat also has another book published without date entitled "Théorie des Poutrées Droites en Fer et Ciment." By a complicated process of analysis Planat arrived at the proportionate depth of the cross-section available for compression. In this he assumes the compressive stress to vary by a rectilinear law. He omits the tensile stresses. His moment is the steel force times the leverage to the centroid of compression. The depth of the compression area he determines to be from $\frac{1}{5}$ to $\frac{1}{3}$ of the depth of the section, depending on the nature of the loading of the beam. Planat appears to have been a competent writer, and his calculations of many forms of construction such as beams, roofs, steeples, reservoirs, retaining walls of cantilever type, were plain and suggestive. He appears to have had extensive knowledge of the physical side of the subject.

Mention should be made of the work of Sanders of Amsterdam in 1898, who assumed a curvilinear elevation between stress and strain on the concrete. Spitzer, of the firm of Wayss & Co., proposed a parabola for this relation in 1897.

It cannot be said, however, that the principles of action had been properly defined and limited prior to 1900. The recognition of the part played by tensile strength of the concrete and the calculation of bond, and of shear reinforcement had yet to be determined upon the basis of laboratory examination. The early constructors, however, were not prevented from applying their systems in successful form throughout Europe, and the researcher into the literature of these early constructions cannot but be impressed with

the apparent completeness with which designers had met the constructive necessities in an economic and skillful manner. Practical constructors like Coignet, Hennebique, Cottancin, Bordenave, Bonna, Matrai, Melan, Wayss, Ransom, Thatcher, built bridges, floors, foundations, aqueducts, factories, silos and retaining walls that showed a general knowledge of the strains to be resisted and a mechanical skill in meeting these strains with economical design that cannot but excite the admiration of present-day constructors. They must have had the elements of faith and imagination to project these structures and also a reliance upon the application of such theoretical knowledge as they possessed.

These early practitioners were, no doubt, without extensive knowledge of the durability of the construction and effect of variations of temperature, but they went ahead and their work was accompanied by test. In fact, if all matters were to be settled before action, very little would be done in this world. Indeed, our own regulations of today, as in case of flat slabs for instance, have been fixed in the absence of a complete knowledge of the manner of functioning of the construction.

It is necessary to call attention to these facts, because those interested mainly in theory, and accustomed to demand a rational process, are likely to undervalue the intuition and mechanical sense of the constructor controlled also by experience and by failures. While the engineers of official bodies doubted and learned persons calculated, these early practitioners, who were in many cases also theorists and calculators, applied their ideas and perfected them; their experiences led them to new departures.

The development of this art on its constructive side was associated with the activities of builders of patented systems. Some of the earliest records exist in the trade publications of these companies.

Monier's simple desire to reduce the thickness of the walls of his pots of concrete led him to the introduction of a metallic network. Monier's first French patent was of date of 1876. This was followed by other patents on reservoirs, floors straight and arches beams. Morsch says that "in his patent drawings are already disclosed all the elements which are today employed in the various construction details of the various systems." This was the point of departure of many applications. Later, in 1884, the rights of the Monier patents were purchased by Freitag and Heidschuh in Neustadt-on-the-Haardt and of Martenstein and Josseaux in Offenbach-on-the-Main. The firm of Wayss & Co., of Berlin, obtained later the rights of the Monier system for the whole of Germany. Experiments and study of the methods of calculations were published by Wayss in the brochure, "Das System Monier," 1887. The firm of Wayss & Co. built many structures throughout Germany and Austria on the Monier system. Monier has often been associated with all forms of reinforced concrete construction. Christophe says that the countries of German language made the fortune of reinforced concrete where it was introduced into construction of buildings and applied to public works.

The sequence of dates below will serve to outline the development. (See Table I.)

TABLE I.—REFERENCES PRIOR TO 1900.

(a) UNITED STATES.

1874–84.	Ransome building various structures, wire rope and hoop iron.
1875.	Concrete house slab and beams, tank tests. Built by Ward.
1878.	Hyatt patent, structures, deformed bar.
1884.	Ransome twisted bar.
1886.	P. H. Jackson's patent.
1888.	Ransome, wine cellar atSt. Helena, Academy of Science.
1889.	Reinforced arch, Golden Gate Park.
1889.	Ransome, Borax works at Alameda (ribbed floor).
1891.	St. Louis Expanded Metal Co's. slabs.
1892.	Leland Stanford University Museum.
1892.	Monier U. S. Patent.
1893.	Monier Arch, Pennypack Creek, Philadelphia.
1893.	Northwestern Expanded Metal Co.
1894.	Haymarket Exchange, Boston.
1894.	Melan Arch, Rock Rapids, Iowa.
1895.	Eden Park, Bridge, Cincinnati.
1898.	Hennebique Patent.
1899.	Thatcher patent.

(b) EUROPE.

1854.	Wilkinson Patent.
1855.	Boat exhibited.
1861.	Coignet book.
1867.	Monier patent.
1868–73.	Reservoirs by Monier.
1869.	Coignet patent.
1871.	Reinforced concrete arch, Ipswich. 50-ft. span, skeleton frame.
1877.	Hyatt's book, theory, tests.
1879.	Hennebique building floors.
1883.	Hennebique, reservoir.
1884.	Monier patents purchased by Freitag and others for South Germany.
1885.	Monier patents, relinquished to Wayss.
1887.	Wayss' book, Das System Monier preceded by tests and theory of Koenen.
1887.	Bordenave construction.
1888.	E. Coignet, theory.
1889.	Cottancin patents.
1890.	Wildegg Bridge, 37.2 m. span, rise 3.5 m.
1892.	Wunsch System, Budapesth.
1892.	Hennebique patent.
1892.	Melan bridges.
1893.	Hennebique girder bridge at Don, France.
1893.	Water conduit at Huesden, Holland, Nolthenius tests, retaining wall.
1894.	Coignet and Tedesco, theory.

1894.	Planat, theory and calculations of retaining walls.
1894.	Moller, of Brunswick, bridges and retaining walls.
1894.	Coignet's project for Alex III bridge.
1895.	Austrian tests on arches.
1892–99.	Hennebique, 3000 constructions.
1896.	Hennebique piles.
1896.	Foundations of church at Brebieves.
1896.	Maryborough bridge, Australia.
1898.	3-hinged arch at Steyer, Austria.
1899.	Chattellerault Bridge.
1887–99.	320 Monier bridges by Wayss.
1899.	Considere and Ritter, theories.

In 1892 the important names of Hennebique and E. Coignet appeared in the history of reinforced concrete. Their designs of beams and bridges extended reinforced concrete to new construction of a monolithic character. Especially is the name of Hennebique prominent in the old applications, such as bridges, retaining walls, stairways, reservoirs, piles, sheet piling. Christophe says that after having worked in France and Germany, Hennebique established branches of his plans in all countries of Europe. In a space of eight years, between 1892 and 1899, Hennebique applied reinforced concrete to 3000 constructions. When one considers the elegance of Hennebique's constructions from the engineering standpoint, he is not so ready to admire the crude efforts of others who received recognition at a later date.

EXAMPLES OF SELECTED EARLY STRUCTURES IN REINFORCED CONCRETE.

1852.—Roof by Fr. Coignet (64 years ago).* (See Fig. 7.)

At St. Denis from three-story brick building. Roof about 12 in. thick, resting on brick walls. Span of slab, 20 ft. Mixture, ½ cement, 1 hydraulic lime, 5 gravel. Reinforcement, H-bars, about 3 in. deep on under side of slab. Both steel and concrete in 1910 were excellent condition. Construction cracks apparently filled up with cement. Roof watertight.

1877.—Thaddeus Hyatt, a lawyer by education, born in New Jersey in 1816, lived most of life in New York and London. The manufacture of his illuminated sidewalk grating was profitable and left him able to devote himself to research. His book in 1877. He died in 1901. His U. S. Patent No. 206112, July 16, 1878. During years preceding Civil War he was active in the antislavery side. Imprisoned for refusing to testify before U. S. Senate in relation to Harper's Ferry incident. Assisted in raising $1,000,000 for relief in Kansas.

1887.—Gas tank.† Height, 5.0 m.; diameter, 12.6 m.; thickness of walls, 13 cm.

Bauschinger's test of Monier Bridge at Munich, 1887. Load, 3824 kg.

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* From *Concrete*, June, 1910. London, Vol. V, No. 6.

† From "Das System Monier," 1887.

per sq. m. Rupture, 4116 on one-half of span and 3252 kg. per 39 m. on the other. Span, 10 m.; rise, 1 m.; thickness, 9.9 to 12.3 cm.; width, 1 m. One mesh at introdos. Total cross-section of resisting bars, 14.9 (cm.)². (Fig. 8.)

Cupola of the Mausoleum of Frederick III at Potsdam. Built by Bonna. Covers, 25 x 37 m. Cantilever brackets project 0.5 m. and carry rolling load of 27,000 kg.

1890.—Foot bridge at Wildegg, Switzerland. Span, 37.22 m.; rise, 3.50 m.; width, 3.9 m.; crown thickness, 0.17 m. at crown; 0.25 at spring. Built by Wayss. Skew, 45 deg. Two meshes of reinforcing. Load, 500 kg. per sq. m.

1890.—Monier Bridge at North German Exposition at Bremen. Span, 40 m.; crown thickness, 0.25 m.; 0.55 m. at spring; rise, 4.5 m.; width,

FIG. 7.—BUILT BY FRANÇOIS COIGNET IN 1852.
ROOF OF HOUSE AT ST. DENIS.

3.0 m. at crown; 8.00 m. at spring. Built in 36 hours. Test load, 1000 kg. per sq. m.

1891.—Bridge near Graeffenhagen. Shown in Rehbein, 1894. Under-crossing for heavy wagon bridges. Span, 2.0 m.; height, 1.5 m. Founded in gravel 1 m. deep, foundation designed with cantilever toe. Reinforced counterfort retaining wall of counterfort type.

1894.—Canal bridge at Draulitten (Canal d'Oberland). Elliptic arch, 26.30 m. span; rise, 6.4 m.; crown thickness, 0.4 m.; at spring, 0.80 m. Reinforced with Monier net, double at spring and single at crown. Design load, 400 kg. per sq. m. Test load, 800 kg. per sq. m. Nine-ton roller.

1896.—Building at Lille. One of the first applications of Hennebique system. Span of girders, 6.80 m.; beams, 4.68 m.; slabs, 1.70; load, 450 to 800 kg. per sq. m.

1896.—Court house at Verviers. Built by Hennebique. Girder, beams and slabs; continuity; stirrups; 500 kg. per sq. m.; girders, 8.9 m. span; beams, 5.10 m.

1897.—Skodsburg Bridge. 21.85 m. span; 2.57 m. rise. Rail reinforcements about 2 per cent at the crown.

1897.—Bridge on Lausanne, Geneva line. Built by Hennebique on skew of 65 deg. Span on skew, 4.25 m.

Bridge at La Rue Valette. (See Marsh, p. 486.) Shows cantilever brackets and slab walls. Steel from barrel into spandrel wall. Retaining

FIG. 8.—TEST OF AN ARCH BY BAUSCHINGER.

wall with relieving arches. Span, 15.0 m.; rise, 1.25 m.; width, 32.8 ft.; load, 28-ton roller.

1899.—Chateaus d'Eua. A remarkable work, built by Ed. Coignet for the Paris Exposition of 1900. It consists of a half cylinder surmounted by a half dome, 144 ft. high. From this niche issues a cascade flowing over stepped slabs supported on beams and radiating arches. Below are passageways and galleries. These are side galleries. In this structure may be found applications of the art in beams, arches, slabs, retaining walls, etc. The face arch has a span of 23.8 m. and is composed in section of a hollow rectangle 2.8 m. deep and 2.0 m. wide with six square bars in the thickening corners. Back of this are two other single arches on the line of the dome. Radiating ribs connect these arches. This admirable work will repay study.

1899.—Chatellerault Bridge. Built by Hennebique in 1899; has a total length of 135 m. There are two arches of 40 m. span and 4 m. rise and a

central arch of 50 m. span and 4.8 m. rise. Sidewalks were cantilevered. The deck, 8 m. wide, is carried by posts spaced 2 m. apart on four arch ribs spaced 2 m. and 0.5 m. wide. These ribs vary in depth from 0.45 m. at the crown to the rock foundation. Piers and abutments are cellular. The design load was 600 kg. per sq. m. of roadway and 400 m. per sidewalk; also a 11-ton truck. The bridge was tested to 800 kg. on the road and 600 kg. on the sidewalk. The deflection was 1/7300 part of the span in end arches and 1/5000 in middle arch. It returned to place without cracks. It was further tested with wagon traffic, marching soldiers, etc., with excellent results. There were no expansion joints. Considere says that the concrete floor, the longitudinal chords and the shortest vertical posts of the Chatellerault bridge are streaked wholly or partly by cracks and fissures, much wider fissures than generally seen in armored constructions.

Hennebique bridges: 1893, 1 girder bridge; 1894, 5 bridges; 1895, 3 bridges; 1896, 10 bridges (girder, 42 ft. span); 1897, 13 bridges (girder, 49 ft.); 1898, 29 bridges (52 ft. span girder); 1899, 42 bridges.

1901.—Bridge at Altare, Italy. Built by Maciachini on Walser-Gerard system. Segmental arch, 59 ft. span and 6.9 ft. rise, 24.93 ft. wide; depth at crown 11.8 in. and 19.7 in. at spring. Cellular spandrel. Reinforced in extrodos and introdos with 0.63 in. rods spaced 7.9 in., tied together. Rods run from ring to foundation of counterfort abutment.

1903.—Pont d'Ivry Bridge. A test span of 65.6 ft. (one-third of the span of the proposed bridge). Two parabolic bowstring reinforced trusses. Girders connected by wind bracing, and a decking supported on beams. Under design load of 60 tons applied for 12 hours, the deflection at center was 1/2000 span. At a load of 180 tons no sign of failure. Failure then began with tension cracks in lower chord. At a load of 220–240 tons the concrete outside the spiral began to scale at a core stress of 5972 lb. per sq. in. A load of 241 tons produced failure.

1894.—Moller retaining walls with arch anchors.

1897.—Cantilever wharf at Chantenay sur Loire. Projects 6 m.; anchored by ties to reinforced earth anchor. Retaining wall, 4.60 m. high between brackets with counterforts, supported on concrete piles.

1898.—Quay wall at Southampton. Hennebique, builder for London and Southampton Railway.

1899.—Retaining wall, Quai Debilly, Paris. Length, 256 m.; height, 5.48 m. to 0. Counterforts spaced, 1.50 m. Projecting platforms at port height with special counterforts. Spaced, 3 m. Well developed toe.

POSSIBILITIES OF UNIT CONCRETE AND STRUCTURAL STEEL AS A MEANS OF MEETING THE SPEED AND ENGINEERING REQUIREMENTS OF MODERN BUILDING CONSTRUCTION.

By Charles D. Watson.*

Mr. Charles E. Fox, of Chicago, the architect who read at the 1915 convention of this Institute a paper entitled "Concrete in Metropolitan Building Construction," very clearly illustrated why reinforced concrete as ordinarily applied has not been generally accepted as fulfilling the conditions required for this class of work. The two most important of these conditions are:

First. A system of construction that insures definite and positive engineering results, which, to quote his words, "will be absolutely safe and secure beyond peradventure, both during construction and after completion, and which prevents failure under any combination of adverse circumstances."

Second. A system that permits of construction in the least possible time.

Mr. Fox illustrated the many advantages which the steel frame had over concrete as regards these requirements, with the idea, as he expressed it, of possibly pointing out the way the concrete frame would have to be developed to meet the conditions of this class of work.

The author numbers himself among those who believe that the form of construction which will finally be accepted as standard for such work will neither be the all-concrete or all-steel frame, but a combination embodying the advantages of both.

Possibly the development of this type of construction has been impeded by the competition between structural steel and concrete, fostered to a certain extent by the conditions under which both have been developed: *i. e.*, engineers specializing in one or the other and competing commercial interests. These conditions are rapidly changing, now that design and construction methods have been so standardized that engineers and architects no longer have to look to experts to handle the ordinary design of either. Such conditions should lead to broader views in the selection of construction materials. We have already seen a start in this direction by the acceptance of the structural steel column practically as a standard for the lower floors of tall buildings with structural concrete frames. Why not continue this application as long as the engineering advantages can be maintained with economy and increased speed of construction? It is, after all, but an engineering detail as to the section, quantity and arrangement of steel which is a requisite of both types of construction.

It is the author's belief that through a combination of structural steel,

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precast concrete units, and concrete cast in place, that the construction requirements of this class of work can best be met, and having recently completed a building in which some of the conditions were analogous to those described by Mr. Fox, he offers this description of the work as a suggestion, that this or, at least, a similar type of construction, is susceptible of development to meet such conditions better than any now in common use.

The work referred to consisted of an addition to the plant of the Syracuse Cold Storage Company at Syracuse, N. Y., the original of which was built in 1909, employing structural steel and a slightly different type of unit concrete designed by the author and previously described before this Institute.

The particular structure herein described is a seven-story and basement building, having about 70,000 sq. ft. of floor space, with three 400-lb., three 300-lb., and one 200-lb. floors, all used for cold storage purposes. The situation which required the construction of this building in the shortest possible time consistent with ecomony was caused by encountering unexpected soil conditions, which prevented the completion of the foundation in the time expected, so that the time allowed for the erection of the super-structure had to be shortened in order to permit the company to fulfill its contracts for acceptance of quantities of perishable produce, requiring the use of all but the two top floors of the building by October 15. The foundations were not completed until August 15, thus giving just two months to get the building under roof and six of the eight floors finished, with the necessary insulation and piping, providing there was no time lost through strikes, delivery of materials or other conditions that commonly prevail in building work. It had been previously decided, for economic reasons, to use the steel frame and unit concrete construction, but three months had been allowed for erection. While there were no delays through labor disputes, the usual weather conditions prevailing throughout the country last year, combined with the inability of the company which had the contract for structural steel to deliver as required, materially increased the difficulty of speeding up the work, the delays in steel deliveries alone amounting to three weeks of the total of two months permissible for such construction.

The odd shape of the building made the application of the unit system unusually difficult and expensive. The ground conditions were quite analagous to a type of work described by Mr. Fox, the building being built out to the street line with only two sides available for handling materials and one of these obstructed by a railway siding which had to be kept open.

The construction consisted of a structural steel frame for columns, girders and wall beams only, the remainder of the structural features being unit concrete sections with brick curtain walls. The panels were approximately 15 ft. square, but less than half the area of each floor had typical panels. The pre-cast beams were made with headed ends to provide sufficient bearing areas on the girders due to the heavy live loads. The beam stems were plain and the slabs were not paneled, being of 2 in. uniform thickness throughout. The fireproofing on the majority of the structure consisted of concrete poured around the lower flanges of the girders to provide a minimum protection of

2 in., the portion between and above the bottom of the beams consisting of pre-cast paneled concrete blocks, the interior columns being reinforced with insulation and plaster as is commonly done in cold storage construction. The beams were spaced about 3 ft. 6 in. on center, and were 15 in. deep with varying stem widths, as well as varying amounts of reinforcement to provide for economical design of floors which had different live loads.

The unit concrete members were all made at a yard temporarily established and within a reasonable hauling distance of the building site. Practically all of the units were delivered to the site by rail, and the freight charges were very little less than they would have been for the average haul from an established factory. Casting was kept thirty days in advance of the requirements of the building. The casting yard covered an area 90 x 250 ft., one end of which was covered by a single stiff-leg derrick with 60-ft. boom. All casts were made in permanent steel forms so designed that they can be used for casting similar units on other buildings by inserting filler plates in the splice to provide for variation in length of the beams, the width and height being obtained by an adjustment of the form itself. The slabs were made in gang molds of 25 slabs, each cast on a vibrator. In this connection it might be interesting to note that the total form cast on this job, charging one-third of the first cost of the steel forms and all special equipment and changes against the work, amounted to less than 2 cents per sq. ft. of floor area.

The beams were cast on permanent beds, the sides being removed daily, the beams themselves being removed every 48 hours and handled as near as possible to a specified time as a method of testing their strength. The beams had surfaces so smooth that they were almost polished, which permited an excellent finish on the completed structure by merely painting their surface.

The 2-in. slabs, having about 15 sq. ft. area, weighed 390 lb. each.

In erecting the building the concrete beams were placed by the steel erectors simultaneously with the structural steel. The completion of the floors followed so closely the steel erection (in many cases floors were being placed before the erection derrick had been removed) that the building was stiffened by the completed floors. This close following-up of steel with finished floors had the additional advantage of saving all temporary floors as required by building laws.

The erection procedure consisted of:

First. Erecting the steel and concrete beams by the steel erectors.

Second. The laying of the slabs and forming of the fireproofing for the bottom of the girders.

Third. Grouting in of the slabs and pouring of the fireproofing.

Fourth. Setting of fireproofing blocks between the beams and pointing.

The slabs were set in mortar beds and immediately struck to a smooth joint on the under side.

In this particular job the slabs were unloaded from the car onto trucks which carried four slabs. These were elevated by hoist to the floor required and wheeled to the proper location in the floor. This, however, was unusual procedure, requiring special equipment not ordinarily used, as the slabs can be more economically handled by the erection derrick. The laying of the

floors was found to proceed very rapidly, one mason and helper with four laborers to carry the slabs laying between 4000 and 5000 sq. ft. of floor in an 8-hour day.

From four to five days under normal working conditions is sufficient time to erect two floors of steel and concrete beams, or an average of two and one-half days to the floor, and floors can be finished at an equal speed without material increase in cost.

The foundations of the building were completed August 15th, and the structure was under refrigeration on October 15th, or just two months after starting, including a delay of three weeks in the erection of structural steel. The work was done with a single shift of men under unusually severe weather conditions, as any one familiar with the rainy season which prevailed in upper New York State during the past summer will testify.

While the author is quite aware that this is far from a record for time in building construction, it does, he believes, establish a record in cold storage construction where concrete is employed. Practically every floor in this building was under refrigeration, and most of them as low as zero temperatures, within thirty days after the time construction was started on such floor. That this could not possibly be done on field-cast concrete is obvious, as the moisture conditions would prevent subjecting it to such low temperatures at this age with safety. Furthermore, these floors were immediately loaded to capacity and there is not as far as anyone has yet found a single shrinkage crack in the structure.

The author believes that the record of progress on this work shows that this method of construction has possibilities of considerable importance in application to the class of work described by Mr. Fox, and where prompt steel deliveries and organizations trained to the requirements of fast construction are available, as they are in such work.

That it meets the other condition emphasized by Mr. Fox of being a type of construction insuring definite and positive engineering results has been quite adequately demonstrated in the large amount of unit work which has already been executed. Mr. Fox states that the conditions which account for the tremendous increase in speed of construction of the modern building are due to the fact that practically all the component parts are shop fabricated. This system of construction adds concrete to that list. Furthermore, a comparison of cost, even at the present stage of its development, indicates that it has the same possibilities of effecting a reduction in the total cost of building such structures as the perfection of the methods in applying structural steel have, as Mr. Fox illustrated, reduced the cost of that material, so that today a steel frame structure averages 20 per cent better at a cost equal to that of fifteen years ago, irrespective of the enormous increased cost of labor and other materials.

This is shown, the author thinks, by the fact that the structure described, complete, exclusive of foundations, cost less than 7 cents per cu. ft., including frame, floors, walls, windows and roofing.

THE USE OF CONCRETE AT THE STATE FARM AT BRIDGEWATER, MASS.

By Arthur J. Maynard,* and Benjamin Baker.

Perhaps the most interesting features of the concrete work which we have done on the State Farm at Bridgewater, Mass., are the character of the labor used, the conditions under which the work had to be executed, and the economies of construction resulting from using institutional labor.

Labor.—The State Farm, which has furnished all the labor, is an institution in three parts. It combines a hospital for the criminal insane, a state almshouse, and a prison to which are sent many men sentenced to short terms for drunkenness and vagrancy, who furnish most of the labor for our concrete work. The prison population numbers in the hundreds.

Cheap and abundant labor is, therefore, the first of our determining conditions. On some jobs I have hired one or two outsiders as foremen, and sometimes carpenters for making the more complicated forms. But there is small error in saying that all the labor for our concrete work has been furnished by the prison. These unskilled laborers have to be trained to their work and new men have to be kept under instruction to replace those that are continually leaving as their sentences expire.

We began our use of concrete late in 1901, when the use of reinforced concrete was just becoming widespread. As I bring before you in chronological order our main pieces of construction, you will see that in a few of them we made early use of advanced engineering knowledge in this field. The reinforcement system of the concrete water-tower at the State Farm is perhaps a fairly creditable example of our engineering capacity. We built beams of 30-ft. clear span as early as 1906.

But if our work has any strong claim to attention, it is because it shows how much can be done for public institutions of the prison and asylum types by the patient use of the labor of the inmates in simple forms of concrete construction. The entire plant of the State Farm was burned in 1883, six years before I went there as Superintendent of Construction. We have now some 90 acres of buildings and their enclosed grounds. Every building erected since 1901 is of concrete and wholly fireproof. Whether the state would have built such a plant by contract, I cannot say, nor do I know just how the actual cost of these concrete buildings compares with what the cost would have been under contract. But as an indication, I may mention that we built the concrete standpipe for $7000 against the lowest private bid of about $13,000.

Early Flat-Slab Roof.—The first piece of concrete construction at the State Farm was a flat-slab cover of a collecting basin adjoining the filter bed of our water supply. Freeman C. Coffin, the engineer in charge, had

* Superintendent of Construction.

designed a roof of brick arches resting on steel I-beams which were to be supported by brick piers. Great strength was required in this roof because it was to be covered with 3 ft. of earth as protection against sun-heating of the water. The change to a flat slab of concrete was made at my suggestion, which was inspired by what I had seen of concrete work in Paris, during a visit to the Exposition of 1900. With the help of William M. Bailey, of the Expanded Metal Company, a scheme of reinforcement for the slab was worked out, and Mr. Coffin generously agreed to the change of plan. That slab was a crude affair, judged by present standards, but it has held up its 3-ft. layer of earth for 14 years. I am told that it appears to be the earliest flat slab built in this country. This work was done in December, 1900.

Wall Construction.—Our first concrete building, a small two-story affair with a dormitory in the upper story and a smoking and lounging room below, was put up in 1901. This building has a feature common to all our dwellings, a double wall, or two walls with an air-space between. The two walls contain no reinforcing metal, and are tied together only at the door and window openings, and at the ends of the floor and roof beams. In this building the walls were 5 in. thick, and the air-space was 4 in.

This method of building double walls without reinforcing metal has been used repeatedly, even in the three-story almshouse, and the four-story dormitory known as "Beacon Street" in the prison yard. It is simple to build, and seems to be adequate structurally.

A second feature of this first building was our method of casting the walls in 1-ft. courses, one course to a day's work. This gave great economy in form lumber and ease in setting the forms for successive pourings.

The Almshouse.—Our largest building is the Almshouse, Fig. 1, constructed in 1906. This is a three-story building made up of three straight sections joined by two bowed sections, and having circular ends. It is 338 ft. 4 in. long, 42 ft. 8 in. wide, and 32 ft. 6 in. high above grade. The wall up to the level of the first floor consists of two 6-in. walls separated by a 6-in. air space. Above this the walls are 5 in. thick, and the air space is 4 in. The two walls are tied to each other at the floor levels and at the openings.

In this building we used a type of pillar that has proved very satisfactory in many ways. The form for the pillar is made of 27 gage galvanized sheet steel, several lengths of sheet tube 30 in. long being soldered end to end with a slight lap. This metal form is held in place by a very light and simple wood framework. As soon as it has been filled with concrete well tamped about the vertical reinforcing rods, the pillar is capable of supporting the beam forms for the floor above. The sheet metal saves lumber, the greater expense of making wood forms and the greater time required for setting wood forms. The metal remains as a permanent covering of the pillars. It is not beautiful, but it serves the purpose. The only finish is paint.

Other Buildings.—The Industrial Building, two stories high and 225 ft. long, was begun in 1906, after the Almshouse was finished, and the work

was carried into the winter. This building has beams of 30 ft. clear span, the greatest clear span we have reached.

The Workshops, built in 1908, may interest you on account of the design of the roof. As this building shelters our cabinet and carpenter work and the large stock of wood that is required, the possible fire danger to surrounding buildings seemed to demand a fireproof roof, so that if things burned inside there could be no great ascending column of fire that the wind might turn against adjoining buildings. For that reason the roof was made of concrete 5 in. thick. With the roof of concrete, we had to protect the stock of machinery from the dripping of water that would be condensed in cold weather on the inner side of the roof slabs.

As insulation against both cold and heat, the outer roof surfaces were treated in the following way: $\frac{7}{8}$-in. wood battens were nailed to the soft concrete, running with the slope of the roof; on these battens was laid a covering of $\frac{7}{8}$-in. boards, and over the boards a standard asphalt roofing.

FIG. 1.—THE ALMSHOUSE, BUILT IN 1906.

The thin air space between the boarding and the roof slabs has served to prevent entirely any condensation on the inner side of the roof. On the concrete slab backs of the saw-tooth windows, the battens for the board covering run with the slope, so that if the roof leaks, the water will run down the slope instead of being ponded, where it might freeze and bulge up the outer roofing cover. Between each two saw-tooths, the main roof rises to a low ridge, and on this section the battens also run with the slope, but crosswise of the building. Water leaking through the covering of the saw-tooth slabs will therefore flow down to the main roof, and there be carried on a new slope to one side or the other of the main roof. There is no chance for leakage to accumulate anywhere between the roof-slabs and the insulating covering.

Silos.—Three reinforced concrete silos, Fig. 2, 40 ft. high and 18 ft. inside diameter, were built later in 1908. The walls are 8 in. thick for the first 20 ft. upward, and 6 in. above that. The reinforcement is $\frac{1}{2}$-in. steel rods, placed vertically 2 ft. on centers; the horizontal rings are spaced 6 in.

for the first 20 ft. of height, and gradually spaced out to 1 ft. The concrete was cast en bloc. Probably the reinforcement is greater than was necessary.

Standpipe.—Prepared by experience with the silos, we built in 1909 our most important structure from an engineering point of view, the standpipe, Fig. 3, 75 ft. high and 30 ft. inside diameter. The scheme of the reinforcement has two main features. For anchoring the base to the side walls three sets of rods, of $1\frac{1}{4}$-in. diameter were used. These were bent to

FIG. 2.—REINFORCED CONCRETE SILOS.

right angles, but with a curve instead of a sharp angle at the bend. One set extends horizontally 9 ft. into the base and 5 ft. vertically inside the wall. The second set reaches 5 ft. into the base and 9 ft. into the wall, and the third set 3 ft. into the base and 13 ft. upwards within the wall.

The wall reinforcement consists of ring rods resting on a sort of trellis which forms the vertical reinforcement. The vertical trellises are spaced about 12 ft. Each of these vertical reinforcing members or trellises consists of five sections, each 14 ft. long. Each section is built of two lengths

of angle steel, $1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$ in., which are connected by flat bars bolted to the two angle pieces. The opposite ends of successive cross-pieces project about 2 in. beyond the vertical members. This arrangement gives a succession of angles in which the horizontal reinforcing rings are laid.

The rings are equally spaced vertically, and extend over about 6 in.

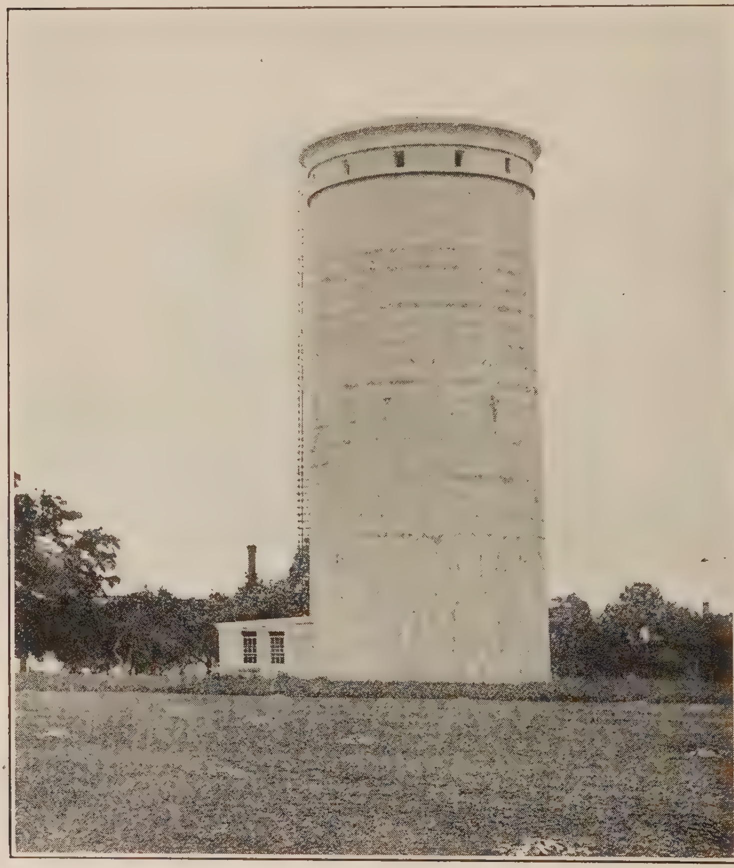

FIG. 3.—REINFORCED CONCRETE STANDPIPE.

horizontally in the lower two-thirds of the wall. In the middle vertical members 1-in. rods are used for the rings instead of the $1\frac{1}{4}$-in. rods of the two lower sections. In the fourth vertical member the rods are $\frac{3}{4}$ in., the vertical angles are 2 in. apart instead of 3 in. as in the lower members, and the vertical spacing of the rings is increased by the steeper angle of the cross-

pieces. In the top members the steepness of the cross-pieces is further increased, and the rings are in consequence further apart.

This arrangement gives at least two advantages. The unskilled workman cannot put the rings anywhere but in the right place, and the arrangement of the metal makes it easy to pack the concrete closely into every corner and angle.

Other Works.—Miscellaneous works in concrete include pig-pens, fences, fence-posts for wire fencing, made in metal forms with the top face naturally flat; drain tile, of which we have turned out thousands of feet; and reinforced concrete walls 23 ft. high.

Terazzo floors, which we use everywhere, may offer a feature of some interest. We have found that a terazzo floor can be laid on spruce plank so that it will crack only at the joint. In the large chapel there are rectangles of terazzo some 30 ft. across without a crack. My method is to lay first a $1\frac{1}{2}$-in. base of cement and sand concrete, one to four, reinforcing this base with expanded metal laid at right angles to the floor joists. The terazzo mixture is laid half an inch thick on this base, which has been scratched with a rake while still soft.

The great heat-resisting powers of Portland cement have been rather notably shown by the linings of one of our large ovens. For the aggregates of this lining concrete I used a lot of firebrick which were pounded down until they provided both fine sand and coarser aggregates. The concrete was made of one part cement, two parts firebrick sand and four parts firebrick chips of about $\frac{3}{4}$-in. screen size. This concrete has endured for several years temperatures often reaching 1000° F. for periods of 10 hours at a time, without any breaking down.

You will have seen that we have used concrete successfully for a great many purposes, working with very crude labor, and with the utmost economy of auxiliary materials. I wish our experience might be an inspiration to the managers of other state institutions. It is certainly possible, with prison and asylum labor, to enlarge the building plant with comfortable and fireproof structures at a very low cost to the public treasury. And I should add that this use of prison labor in out-of-doors work is a decided benefit to the prisoners themselves. In the 26 years of my experience in working prisoners outside of the prison yard, without guards, I have not lost a single man by escape, though on the Almshouse job there were nearly 200 men at work every day for many weeks.

DISCUSSION.

Prof. Hatt. PROF. HATT.—A little more information is desirable concerning the flat slab roof built in 1900. What was its thickness and the arrangement of the wall and slab reinforcement?

Mr. Wilson. MR. WILSON.—The wall space in these buildings is decidedly interesting, and the method used by the author in preventing wet concrete from dropping into them ought to be stated.

Mr. Maynard. MR. MAYNARD.—A description of the early flat-slab roof was printed in several trade papers in February, 1901. The slab was 7 in. thick, reinforced with expanded metal, and covered a circular basin 75 ft. in diameter. At the supports the vertical reinforcement went up into the slab and the expanded metal was laid over it.

The air space in the walls was covered when the concrete was poured; we used a sort of wedge device which is readily withdrawn. There are no ties of any sort between the walls, because the spacing between windows is seldom more than 6 ft. and ties do not seem to be necessary under such conditions. The small wires that hold the forms are left in place, it is true, but it is doubtful if they have any value as ties.

President Wason. PRESIDENT WASON.—One of the most interesting things to be seen at the State Farm is the oven, referred to in the paper. I saw it after it had been in use several years. The roof was then red-hot over the fire, which was within a foot of the concrete. This exposure to intense heat had been going on for a long time and yet the roof had shown no signs of injury. This experience indicates the value of crushed firebrick as an aggregate for concrete subjected to high temperatures.

(The paper was followed by the presentation of a number of lantern slides, during which there was considerable discussion which could not be reported on account of the darkness.)

THE MIDDLEBORO, MASS., REINFORCED CONCRETE WATER TOWER TANK.

By G. A. Sampson.*

The history of reinforced concrete standpipes and elevated tanks for storage of water had its origin in this country at Milford, Ohio, in 1903. The first structure was a standpipe having a capacity of 93,000 gal. with an inside diameter of 14 ft. and a depth of water of 78 ft. Although the development in this branch of concrete construction has been somewhat slow in the thirteen years elapsed, and marked with a few doubtful successes, there are nevertheless a considerable number now in use that have proven the adaptability and advantages of concrete for these types of structures. From the experience acquired through the thirty or more standpipes of particular note and the comparatively few elevated tanks, considerable progress has been made in fundamental design, although at the present time there are many important points which require further study in order to establish this subject in the field of recognized general practice. It is the aim of this paper to briefly present the main features of design and construction of the Middleboro Tower Tank, the largest elevated water tank in the United States and the first to be built with a hemispherical bottom.

Foundation for Tower.

The foundation for the tower is a circular ring in plan with a width of 15 ft. and an area of 1970 sq. ft. It extends 7 ft. below the finished grade and rests upon a very fine, well-compacted sand, the safe bearing power of which was determined experimentally by loading a timber set at the elevation of the bottom of the foundation. At a loading of from 1 to $3\frac{1}{2}$ tons per sq. ft., the settlement was uniform and slight, being only $\frac{1}{32}$ in. for each additional one-half ton; but 4 tons per sq. ft. produced a relatively large settlement, $\frac{5}{32}$ in. for the last one-half ton.

The footing area is such that the total dead weight of 4485 tons compresses the soil to the extent of 2.28 tons per sq. ft. A gale of 70 miles per hour, the equivalent of a horizontal pressure of 30 lb. per sq. ft. on a vertical projection of the structure, produces a maximum compression at the outer edge of the foundation of 0.68 ton, thereby giving a total maximum load of 2.96 tons per sq. ft. Although the resultant of the wind pressure is slightly more than doubled with an empty tank, it is more than offset by the reduction in the dead weight of the water, so that the maximum mentioned represents the worst condition.

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TOWER.

A hollow cylindrical tower was adopted in preference to a number of columns, because structurally it seemed a safer type of support, would probably be more economical to construct and could be designed to present a more pleasing appearance. Considerable study was given to the architectural side of the problem, in order to produce at a moderate cost a structure that would be an addition to the landscape. A concrete seat encircling the base of the tower accomplishes the appearance of solidity. The plain aspect of a simple cylinder is relieved by twelve 4 x 24-in. pilasters. A balcony of reinforced concrete 108 ft. above the ground, with bracket supports, paneled posts and railing furnishes a suitable vantage point for inspecting the outside of the tank and provides a visual starting point for the tank proper.

The thickness of the tower walls between pilasters is 10 in. and the greatest compression due to the dead load is 483 lb. per sq. in., increased by a 70-mile gale to 631 lb. Entrance to the tower is through an iron door and light and ventilation are obtained by means of twelve windows, three of which are arranged to open. An iron ladder within the tower leads from the ground to the balcony through a small door in the wall of the tower at the elevation of the balcony floor. Another iron ladder on the outside of the tank connects the balcony with the roof above.

A 16-in. cast iron flanged supply pipe, fitted with an expansion joint, rises from the ground and enters the bottom of the tank at its center. An 8-in. cast iron flanged overflow and drain pipe discharges at a safe distance from the base of the tower.

TANK.

The tank proper, 41 ft. in inside diameter and having a depth of water at the center of 59 ft., has a capacity of slightly over half a million gallons. An interesting feature is the bottom of the tank, which is a hemispherical bowl hung at its rim from the wall of the tower. The bowl itself has a capacity of 125,000 gal. The advantages of this type of bottom over the flat beam and slab construction commonly employed for small tanks, and the dome type used in a few structures of considerable size, are real enough to warrant careful consideration in the design of elevated concrete tanks. As compared with the dome bottom, there is a saving of 34 per cent in the height of the vertical wall of the tank for the same capacity, or a 47 per cent increase in capacity for the same height of wall, a practically self-cleaning bottom and a better distribution of stresses because the enormous thrust in the dome bottom, which is concentrated at the most critical point of the tank, is replaced by a uniform tensile stress coincident with and analogous to the tension in the tank wall.

The vertical wall of the tank varies in thickness from 10 in. at the top to 16 in. at the bottom and the hemispherical bottom from 18 in. at its connection with the tank wall to 14 in. at its center. The thickness of concrete is such that without any assistance from the steel reinforcement its stress in tension is about 250 lb. per sq. in. The tensile stress in the steel acting

independently is approximately 14,000 lb. per sq. in., and with both materials acting in conjunction the computed stresses are 215 and 2150 lb. The tank is covered to guard against the growth of algæ, not uncommon in a filtered water supply exposed to light, and to prevent freezing. The roof is a concrete dome 4 in. thick, 41 ft. in diameter and with a rise of 4 ft. at the center.

CONCRETE.

Samples of cement were taken as loaded on the cars, which were then sealed. Sampling and testing conformed to the specifications of the American Society for Testing Materials. The results of the twelve carloads required, aggregating 8100 bags, is given in Table I.

TABLE I.—QUALITY OF CEMENT USED IN MIDDLEBORO WATER TOWER.

No. of Carload.	No. of Bags.	Tensile Strength.					Time of Setting.		Fineness, Per Cent Residue.		Specific Gravity.
		Neat.			1 : 3 Sand.						
		24 Hrs.	7 Days.	28 Days.	7 Days.	28 Days.	Initial, Hr. Min.	Final, Hr. Min.	On No. 100 Sieve.	On No. 200 Sieve.	
1	700	323	643	704	379	454	3–00	6–10	3.0	19.6	3.18
2	700	330	691	779	406	441	3–05	6–30	3.2	19.8	3.17
3	700	329	698	712	398	485	3–35	6–30	3.6	18.6	3.17
4	700	382	627	703	366	465	4–20	7–00	3.8	20.4	3.17
5	700	481	712	715	315	354	3–53	5–08	3.8	20.8	3.15
6	700	383	593	710	343	402	3–15	5–50	3.6	21.8	3.15
7	700	330	768	820	364	458	3–06	5–31	3.2	20.2	3.16
8	700	362	753	787	357	434	2–43	5–28	3.2	20.0	3.16
9	700	302	737	687	375	419	3–34	6–19	3.4	20.8	3.17
10	800	325	651	747	305	416	3–46	5–31	3.2	21.2	3.16
11	600	374	697	705	259	425	3–44	5–29	3.4	21.6	3.16
12	400	484	759	763	340	455	4–00	6–45	3.4	21.0	3.15
Total and Average	8100	367	694	736	351	434	3–30	6–01	3.4	20.5	3.16
Requirements		175	500	600	200	275	Not less than ½ hour.	1 to 10 hours	Not over 8.0	Not over 25.0	Not less than 3.10

The sand and coarse aggregate were obtained from a gravel pit about 1¼ miles from the work and were screened by hand to give the required sizes. They were found by tests to be so well graded for maximum density that it was not necessary to add any other sand or to separate the stone into various sizes for the purpose of combining them in other proportions. The sand, which contained about 3 per cent of clay, was tested for tensile strength compared with standard Ottawa sand, and gave an average of 112½ per cent thereof against 100 per cent specified. The gravel stones were washed by hose stream at the pit to remove a slight coating of clay. The proportions of the concrete for the foundation are 1 of cement, 2½ of sand and 5 of gravel stone, and for the tower, balcony, roof and upper 4 ft. of the tank, 1:2:4.

For the hemispherical bottom and lower 23 ft. of the wall of the tank, the ratio of cement to the sum of the aggregates was fixed at 1: 3. Mechanical analyses of the ingredients were made and the proportions of sand and stone to produce a concrete of maximum density were first studied by the methods proposed by Fuller and Thompson. Finally actual mixtures of various proportions were made, using a constant weight of water and total aggregates, and carefully molded in an iron cylinder 6 in. in diameter and 24 in. in length. The mix of greatest density, as determined by the space occupied in the cylinder, was found to be $1:\frac{7}{8}:2\frac{1}{8}$ by volume, which was the one adopted. The following 14 ft. of the tank wall were mixed in the proportion of $1:1\frac{1}{2}:3$.

The maximum diameter of stone allowed in the foundation was $2\frac{1}{2}$ in., in the tower and tank 2 in. and in the roof $1\frac{1}{2}$ in. All concrete was mixed in a 15 cu. ft. Smith mixer operating at 15 revolutions per minute, and the minimum time of turning varied from $1\frac{1}{2}$ minutes for the foundation to about 2 minutes for the tank. A slight increase in homogeneity was noticed up to 4 minutes, but this latter period was not required, as the various operations of placing produced a comparable result. The concrete materials were brought to the mixer in dump cars running on an incline industrial railway, and pulled up the incline by a cable attached to a drum of the hoisting engine. The mixer discharged into a bucket elevator in a 225-ft. tower, which tripped at the desired height and dumped into a receiving hopper, whence it was conveyed by gravity through a sheet steel chute to a central distributing hopper and finally by means of eight movable wooden chutes was deposited between the forms around the circumference.

The writer was converted to this method of placing concrete somewhat against his will, but was finally convinced that with due care in mixing, in the addition of water, in the angle of inclination of the chutes and finally in placing and spading the concrete between the forms, a concrete can be produced equal to that secured by any other practical means. The structure contains 1120 cu. yd. of concrete.

STEEL REINFORCEMENT.

Round, deformed, open hearth, hard grade reinforcing rods bent at the rolling mills, were used in the structure. These were tested at the Carnegie mills by the New England Bureau of Tests; a large proportion of them were $1\frac{1}{4}$ in. and developed an ultimate tensile strength of 120,000 lb. per rod. Laps were 40 diameters and in addition each joint in the tank was secured by two cable clips. The horizontal rods of the tank were firmly secured in their true position to 16 vertical 3-in. channels drilled to the exact spacing of the rods. There were 160,000 lb. of steel reinforcement embedded in the concrete. The average results of tests of the round bars are given in Table II.

FORMS.

Steel forms of No. 16 gage metal, stiffened horizontally and vertically with 2-in. angles, and furnished by the Blaw Steel Construction Company of New York, were used for the outside of the tower and tank. Two complete

rings were provided, each 4 ft. in height, at a cost of \$600. Two sets of wooden forms of the same height, costing \$400, were used on the inside surfaces. Some difficulty was experienced at the beginning in holding the steel forms sufficiently rigid in the vertical bolted joints to prevent a slight increase in circumference when loaded, but this movement was prevented by $\frac{7}{8}$ in. wire cables placed near the top and bottom of the ring to be concreted and subjected to an initial stress by means of turnbuckles.

The use of steel forms was fully justified, both on account of the resulting smooth, dense surface requiring practically no finishing, and on account of a considerable saving in expense of erection. It required from five to six hours to remove, erect, line up and brace a ring of inside wooden forms and about six hours to remove and erect a set of outside steel forms, including the pilaster sections.

TABLE II.—AVERAGE RESULTS OF TESTS OF BARS USED IN MIDDLEBORO WATER TOWER.

Diameter, in.	Elastic Limit, lb. per sq. in.	Ultimate Strength, lb. per sq. in.	Elongation in 8 in., per cent.	Chemical Analysis.			
				Carbon, per cent.	Manganese, per cent.	Phosphorus, per cent.	Sulfur, per cent.
$1\frac{1}{4}$	50,880	89,670	17	0.48	0.69	0.014	0.033
$1\frac{1}{8}$	53,890	87,780	21	0.42	0.72	0.012	0.033
1	53,860	93,860	21	0.50	0.79	0.014	0.040
$\frac{7}{8}$	54,820	91,420	14	0.47	0.71	0.031	0.050
$\frac{3}{4}$	57,410	102,100	15	0.52	0.68	0.015	0.040
$\frac{1}{2}$	54,440	87,600	18	0.46	0.69	0.014	0.036
$\frac{3}{8}$	57,000	88,000	19	0.43	0.70	0.011	0.029

The hemispherical bottom and dome roof were cast between forms consisting of wooden ribs covered with sheet steel, with very satisfactory results.

JOINT BETWEEN OLD AND NEW CONCRETE.

For the tower, in order to secure a bond between the successive rings of concrete, the forms were overfilled about $\frac{1}{4}$ in., which surplus containing the laitance was screeded off previous to the initial set of the concrete, following which, as soon as the final set occurred, the surface was wire brushed to slightly expose the stones, and immediately before placing fresh concrete washed with a hose stream and coated with a neat cement grout. For the tank, in addition to the above precautions which were carried out with especial care, there was cast in the old concrete a continuous triangular groove, about $1\frac{1}{2}$ in. in depth, running around the wall near its center and subsequently filled with grout and concrete; to still further minimize the chances of leakage at each horizontal joint an uncoated steel plate or dam of No. 14 gage metal 10 in. wide, turned over 1 in. at each edge to form a channel and bolted together to form a continuous water tight stop, was embedded 4 in. into the old work, thus leaving 4 in. extending up for a bond with the new concrete. The cost of 12 channel rings in place was about \$400.

PROGRESS OF WORK.

Ground for the foundation was broken on April 26, 1915. After the excavation was completed the earth was wetted and thoroughly compacted by rammers, following which a 2-in. layer of concrete was placed over the entire surface to serve as a working base for the erection of the steel reinforcement and as a suitable medium upon which to place concrete.

The average rate of progress on the tower was a 4-ft. ring each alternate day, with a marked increase in efficiency as the work advanced. The actual time required for concreting was about 4 hours to each 4 ft. section. The bottom of the tank, including 4 ft. of the wall immediately above the tower was poured continuously. Concreting began at 5 A. M. on September 9th and the last batch was deposited at 4 A. M. the following day, during which time about 130 cu. yd. of 1:1:2 concrete were placed. Five days elapsed before the following section was in place and two more days before the succeeding ring was concreted. Four days were then spent in preparing for future operations, so that when concreting began again on the fifth day the following eight successive lifts were made in nine days and it was only due to a violent gale of about 70 miles per hour that operations had to be suspended for one day. The roof of the tank was poured on October 13th.

TREATMENT OF CONCRETE SURFACES.

At the time of the removal of the forms from the inside of the tank and the working staging from around the outside, the surfaces were cleaned and so far have received no further treatment, although after the tank has been fully tested and accepted as satisfactorily watertight, the question of waterproofing the inside and damp-proofing the outside surface will be considered. If the final tests are as satisfactory as the results so far obtained, the expense of waterproofing would hardly be justified. Coincident with the removal of the outside staging around the tower, the surface was cleaned and washed with a brush coat of neat cement grout, mixed in proportion of 1 of cement to 1 of water. The cost of cleaning and coating the 14,000 sq. ft. of surface was $75, or about 0.54 cents per sq. ft.

WATERTIGHTNESS.

In connection with the subject of watertightness, the following clause from the specifications will be of interest:

"The concrete in the bottom and wall of the tank shall be substantially watertight at all times. Any leakage amounting to jets or visible seepage shall be repaired by and at the expense of the contractor and by methods approved by the engineer. Small damp spots, if few in number, which do not increase nor disfigure the appearance of the concrete, will be considered to meet the requirements hereunder. It is intended that the work shall be sufficiently waterproof that severe freezing will not at any future time threaten the integrity of the concrete and that the general appearance of the structure will not suffer through efflouresence or other disfiguring stains."

Water was let into the tank on October 24th and the depth gradually

increased until on October 30th, with 26 ft. of water, three hair-cracks developed, radiating from the central 16-in. inlet main and extending 6 to 9 ft. therefrom. The water was drawn off and a circular reinforced concrete ring 9 in. thick, bonded to the old bottom by roughening, grouting and doweling, was cast on the inside of the tank around the pipe. The structure was permanently put into service on January 1, 1916, and although the present depth of water is only about 30 ft., no weakness or leakage has developed and we anticipate no further trouble from this source.

The sole reason for the cracks was undoubtedly insufficient steel reinforcement in the design and does not reflect in any way on the type of bottom or the workmanship of the contractor. The vertical wall of the tank, with the exception of three or four very small damp spots on one horizontal joint where an excessively hot day caused shrinkage cracks in the top of the ring

TABLE III.—UNIT COSTS OF MIDDLEBORO WATER TOWER.

Item.	Quantity.	Cost.			Unit. Cost.
		Materials and Supplies.	Labor.	Total.	
Earth excavation	600 cu. yd.	....	$267.99	$267.99	$0.45
Backfill and grading	600 "	....	225.42	225.42	.38
Concrete (excluding reinforcement):					
Foundation	176 "	$776.28	573.47	1,349.75	7.67
Base of tower	58* "	333.51	445.65	779.16	13.43
Tower	450 "	2,907.70	4,954.00	7,861.70	17.47
Balcony	48 "	289.53	556.65	846.18	17.63
Hemispherical bowl	108 "	1,279.47	2,236.54	3,516.01	32.56
Wall of tank	244† "	1,782.32	3,036.98	4,819.30	19.75
Roof of tank	17 "	186.50	385.47	571.97	33.65
Seat	4 "	46.83	100.64	147.47	36.87
Belting course	14 "	115.09	237.14	352.23	25.16
Steel reinforcement	160,000 lb.	2,828.00	1,103.95	3,931.95	.0246

* 36 cu. yd., 1 : 2½ : 5; 22 cu. yd., 1 : 2 : 4.

† 16 cu. yd., 1 : 2 : 4; 57 cu. yd., 1 : 1½ : 3; 171 cu. yd., 1 : 1 : 2.

poured on the previous day, has been absolutely tight, as well as the hemispherical bottom, except for the above mentioned cracks.

PRICES AND COSTS.

Prices of standard materials of construction have advanced so sharply during the last few months and local prices of other materials entering into any work are so variable, that in order to render the following costs of any particular value it is necessary to state the prevailing prices at the time the work was done. Cement in cloth bags, f.o.b. cars Middleboro, was $1.45 per barrel and the 1½ mile haul cost 10 cents additional; sand was delivered at the work under a sub-contract for 85 cents per cu. yd., and gravel stone, screened and washed, for $1.85 per cu. yd. Steel reinforcement, cut to length and bent to the required radii, cost 1.7 cents per pound, f.o.b. Middleboro and the hauling amounted to 50 cents per ton. Lumber for forms, staging, towers,

etc., averaged \$28 per 1000 ft. B. M. Unskilled labor was \$2.00 and \$2.25 per 9 hour day, and skilled labor from 45 to 60 cents per hour.

The cost to the contractor of some of the principal items of the work are given in Table III. These costs include superintendence, liability insurance, buildings and all other expense except general overhead charges, such as traveling expenses, freight, repair and depreciation to plant, blacksmithing, miscellaneous teaming and small supplies which would add about 10 per cent to the figures.

The average cost of concrete per cubic yard, exclusive of reinforcement and general overhead charges, was....................	\$18.09
The average cost of concrete per cubic yard, including reinforcement, but excluding general overhead charges, was.........	21.60
Total cost of the completed structure to the town, including engineering, was....................................	27,500.00

The contractor was the Hennebique Construction Company of New York. Mr. Alvin C. Howes is chairman of the Middleboro Water Commissioners, and Thomas F. Dorsey was Resident Engineer for the writer, under whose direction the work was designed and constructed.

DISCUSSION.

Mr. Wilson.—In some large concrete tanks at San Francisco, which have flat bottoms, much trouble has been experienced at the junction of the walls and the bottoms. It seems to me that there might be the same trouble in the Middleboro tank at the junction of the hemispherical bottom with the vertical walls, due to the various stresses acting at this place. **Mr. Wilson.**

Mr. Cobb.—In view of the trouble I have experienced at the junction of a roof with side walls, due to expansion and contraction there, I think the author should tell us something about his experience with this detail. **Mr. Cobb.**

Mr. Sampson.—One of the advantages of using a hemispherial bottom is that it does not develop such stresses as exist where a flat bottom is employed. Both the wall and the rim of the hemisphere are equally stressed and the stresses are provided for in the same way, so that there is no such tendency for the wall to pull away from the bottom as exists with a flat bottom. At the junction of the bottom and the wall the load is all downward and easily calculated, and provision for meeting it is made by carrying the radial rods of the hemispherical bottom vertically upward in the wall. In other words, the bottom is hung from the wall. As the temperature of the water pumped into the tank does not vary much more than 10 deg. during the year, and the tank is roofed over, it does not seem probable that temperature stresses are likely to be of any appreciable amount and no attention was paid to them in the design. **Mr. Sampson.**

SOME FEATURES OF CONCRETE WORK ON SUBWAY CONSTRUCTION, NEW YORK CITY.

By Robert Ridgway.*

The Public Service Commission for the First District, State of New York, is directing the construction and equipment of a comprehensive system of rapid transit for New York City. This is known as the Dual System, because when completed its two divisions will be operated for a term of 49 years by the Interborough Rapid Transit Company and by the New York Consolidated Railroad Company as assignees of the New York Municipal Railway Corporation, under leases covered in the instruments known as Contracts No. 3 and 4, which were signed on March 19, 1913. The New York Municipal Railway Corporation is a subsidiary of the Brooklyn Rapid Transit Company.

The proposed system is an extension of existing rapid transit elevated and subway lines, and will serve four of the five boroughs of the city. It means an increase in single-track mileage of the present rapid transit lines, subway and elevated, from 296 to 620 miles; and it is estimated that the existing transportation facilities will be more than trebled. The scope of this paper does not permit a description in detail of the location and connections of the various routes proposed and under construction, but the following figures will give an idea of the importance and magnitude of the project.

The estimated cost of construction and equipment is approximately $330,000,000, of which the city contributes about $164,000,000, the Interborough Rapid Transit Company about $105,000,000, and the Brooklyn Rapid Transit Company about $61,000,000. These figures include the amounts to be spent by the two companies for extending and third-tracking their present elevated railroad lines. Of the city-owned lines, contracts of a total approximate value of $163,000,000 have been let, and the value of the construction work accomplished to date is about $109,000,000. Some of the lines are ready in operation and the bulk of the construction work will be finished in 1917.

The plans provide for many types of structures, such as subways constructed in open cut and in excavation under decking, soft-ground and rock tunnels, subaqueous tunnels driven with and without the use of shields (some of them under compressed air), subaqueous tunnels of the Detroit-River type, and elevated railway construction of steel and reinforced concrete. The underpinning of buildings from one to twenty stories in height is an important feature of the subway construction, especially in downtown Manhattan. In a few cases, the subway will pass directly under existing buildings, the most notable instance of this being the U. S. Post Office and Court

* Engineer of Subway Construction, State of New York, Public Service Commission for the First District.

House Building, under which a two-track subway is to pass between Broadway and Park Row, Manhattan. In other cases where the subway passes under private property, the structure is designed to support future skyscrapers.

Approximately 3,000,000 cu. yd. of concrete will be required in the construction of the city-owned subway and elevated lines, of which 1,600,000 cu. yd. had been placed prior to January 1, 1916, and 425,000 cu. yd. during the year 1915. Of an estimated total of 5,500,000 bbl. of cement required for concrete and other classes of masonry for the same lines, 3,000,000 bbl. had been inspected prior to January 1, 1916, and 1,206,000 bbl. during 1915. All cement and concrete aggregates, as well as all other materials entering into permanent construction, must be inspected and approved by the Division of Materials Inspection of the Commission before it can be used in the work.

INSPECTION OF CEMENT AND CONCRETE AGGREGATES.

It has been the general policy of the Commission to maintain its own inspection force and laboratories, a policy which experience had amply justified. It would have been a serious mistake, in the author's judgment, to follow any other course on a work of such magnitude as the one under consideration. Since the beginning of subway construction in 1900, a laboratory has been maintained in Allentown, Pa., which controls the testing of cement. A long step in advance was taken about two years ago when a physical laboratory was established in New York City to test concrete aggregates, as well as samples of concrete mixed on the work under service conditions. The cost of this laboratory is justified by the size of the work and the results which have been obtained.

The advantage of knowing, by physical tests conducted on scientific lines, the quality of the concrete which is actually being placed in a work conducted under such varying conditions as obtain here, is too apparent to need pointing out. A study of the results of the tests indicates a more uniform and better quality of concrete than was obtained before the tests were undertaken. What is believed to be a departure from usual practice is the assignment of inspectors to the plants where the concrete aggregates (sand, gravel and broken stone) are produced, thus controlling the grading and quality of these materials. Engineers generally are now alive to the importance of testing the aggregates and controlling their quality, a matter which has received too little attention in the past, as the opinion prevailed in many quarters that if a good quality of cement was obtained, the quality of the aggregates was of relatively minor importance. Undoubtedly the failures of some concrete structures have been caused by the use of poor aggregates.

In connection with the testing of concrete aggregates, the author would like to quote a memorandum prepared by Mr. Ralph E. Goodwin, Junior Engineer, in immediate charge of the Commission's physical laboratory, under Mr. George L. Lucas, General Inspector of Materials, regarding the inspection of concrete aggregates:

"The extensive construction of new rapid transit railroads has put an unusual demand on the market for sand, gravel and broken stone for con-

crete. This heavy demand, coupled with keen competition between different dealers for the new business, early made it imperative to devise standard methods of inspection, in order both that all the dealers should meet uniform requirements and to safeguard the quality of the material. The quantities of aggregates used are so large that it has been found practicable and advisable to locate inspectors at the producing plants. The material is loaded in scow-load lots under the supervision of the inspector, and for each accepted load a ticket is issued signed with the inspector's name to facilitate identification at its destination.

"As the inspectors are located at points from 25 to 50 miles distant from the work, and are sometimes out of touch with the main office, it has been found necessary to interpret the general specifications for them and to formulate definite rules for their guidance. The general specifications state that all aggregates must be clean and well graded, but do not give any limiting percentages, such as are essential to insure uniformity of inspection by different inspectors at separate plants.

"Before deciding upon such limiting percentages, a search of concrete literature was made for suggestions as to the best method to pursue, and the methods adopted are substantially the same as those used by Mr. Fuller some years ago in his experiments for the Aqueduct Commission of New York City. The size of gravel and stone is tested by screening a sample through a nest of screens mounted in an agitating apparatus, and the size of sand is similarly tested in a nest of sieves. Both sand and gravel must be free from loam and clay. All inspectors are equipped with the screens, sieves, weighing scales, silt tubes and other apparatus necessary for making the tests.

"The grading of all aggregates is controlled by percentage limits as to the amounts permitted to pass each size of sieve. A maximum and minimum figure is given for each size, thus allowing a reasonable variation, such as is to be expected under commercial conditions. By recognizing the fact that some variation is unavoidable, and providing for it in the specifications, disputes are practically eliminated. The advantages of this method of specification writing should be more generally recognized.

"The limits were chosen as follows: For gravel and stone the ideal grading was assumed to be a straight line analysis from $\frac{3}{8}$ in. size to $1\frac{3}{4}$ in. The upper size is the maximum which is satisfactory for general use on the work on account of reinforcement and steel members, while the lower limit is necessary because small gravel or stone has a great tendency to segregate from the larger sizes. On this account it is almost impossible to include the sizes between the No. 8 sieve and $\frac{3}{8}$-in. stone or gravel, though these sizes are desirable from the standpoint of maximum density. Another practical consideration, which is an argument against including any large amount of small material in coarse aggregate, is that the smaller the size of the material, the more dirt accompanies it, while only a small amount of dirt accompanies material screened over large holes.

"Between the upper and lower limits specified, the maximum permissible variation from a straight line analysis was arrived at by taking into

consideration the character of the best material obtainable and the effect of variations in grading as shown by test. The average engineer probably overestimates the importance of using carefully graded coarse aggregate, since the fact is generally known that laboratory tests show carefully graded aggregate to produce the strongest concrete. But the small effect of variations from ideal grading is sometimes overlooked. Grading reduces the voids by not over 5 or 10 per cent and the effect on the strength of a rich mixture is not so great as are the effects of variations in measurement of materials in the field, consistency, length of time mixed and care in placing. In the field, also, graded aggregates suffer more or less segregation, so that frequently the actual grading of materials placed in the mixer is quite different from that worked out in the laboratory. In fact, the principal advantage of using graded coarse aggregate is that it is easier to handle and produces a somewhat smoother-working concrete. The latter feature is not alone of advantage to the contractor by reducing his costs, for we should never lose sight of the fact that any factor which makes a smoother working concrete and gives a better appearing job, also increases the strength by reducing honeycomb and increasing density. There is danger in a too literal interpretation of laboratory results, as field and laboratory conditions are vastly different, and the best practice is that which gives good results in the field as well as in the laboratory.

"The limits for the mechanical grading of sand were arrived at by taking into consideration Mr. Fuller's curve for maximum density; but his ideal sand was found to be too coarse to make a concrete which would hold together, and his curve required modification. For the lower limit of size it was attempted to specify percentages which would eliminate sands of such fineness as to make them inferior to Ottawa testing sand, as shown by compression tests. The percentage limits for the mechanical grading of fine and coarse aggregates are shown in Table I. It is important to note that the size of aggregate made by the commercial rotary screen is not the same as the size of hole through which it passes, on account of the fact that the size of material passing each screen depends not alone on the size of hole, but also on the speed, slope and length of the screens, and on the rate of feed. The figures in Table I refer to the actual size of aggregate as determined by tests of samples with testing screens.

"Inasmuch as inspectors supervise the loading of all concrete aggregates used on subway work at the plants where they are produced, it has been possible to adopt a new method of handling at such plants as possess suitable facilities. This new method consists of mixing the sand and the coarse aggregate together in the proper proportions on the scow, so that in order to make concrete it is only necessary to add cement to the mixture of aggregates, thus avoiding the inconvenience of storing and handling two separate aggregates on the work. A further advantage of this method is that it permits the small size of gravel usually called "grits" to be included in the mixture without segregation, thus increasing the density and strength of the concrete. This mixture of fine and coarse aggregates has given satisfaction when carefully handled, but is only used on a few of the many subway contracts. If

not carefully handled the fine and coarse aggregates show a tendency to separate."

Regarding the organization of the physical laboratory and results obtained there, the author refers to articles prepared by Mr. Goodwin which were published in the following papers: *Engineering Record*, January 8, 1916; *Engineering News*, February 4–11 and 18, 1915; *Cement Age*, November, 1915; *Concrete*, November, 1915. Mr. Goodwin has done much to make the laboratory a success, and his work has attracted the favorable attention of those experts who are familiar with what he is doing.

Among tests which he has conducted are those of the watertight qualities of concrete. His results indicate that properly mixed and properly proportioned concrete, without the admixture of waterproofing liquids or compounds,

TABLE I.—PERCENTAGE LIMITS FOR MECHANICAL GRADING OF FINE AND COARSE AGGREGATES.

FINE AGGREGATE.		LIMIT OF	
Size of Opening, in.	Sieve Number (square holes).	Fineness (not more may pass).	Coarseness (not less may pass).
0.20	4	100	95
0.10	8	95	85
0.042	16	75	40
0.021	30	50	20
0.011	50	30	2
0.006	100	6	..
COARSE AGGREGATE. Screen Number (square holes), in.			
2		..	100
$1\frac{1}{2}$		..	95
1		80	40
$\frac{3}{4}$		60	25
$\frac{1}{2}$		40	10
$\frac{1}{4}$		5	..

can be made as water-tight as concrete mixed with such materials. His results bear out those of the U. S. Government tests referred to in "Technologic Bulletin No. 3," U. S. Bureau of Standards.

MIXING AND PLACING CONCRETE.

The mixing and placing of concrete on the work follow standard practice, and the methods generally are not novel. Where work has to be carried on under such conditions as obtain in crowded city streets, it is obvious that the cost cannot compare favorably with that of work conducted under less trying conditions. The safety and convenience of the public must be considered before economy, and there is lacking, therefore, some of the incentive to devise new and less expensive methods of doing the work.

The standard form of contract contains this specification:

"Whenever practicable, concrete shall be machine mixed. A rotary machine of a pattern approved by the engineer, and mixing only one batch at a time, shall be used. Concrete shall not be mixed on the street surface on the line of the work, unless specifically permitted."

Many standard types of rotary batch mixers are successfully used along the line of the work. It is believed they furnish a more uniform quality of good concrete than can be obtained by any other type. On one or two of the large contracts, and in special cases on other contracts, the concrete is hand mixed, this being permitted by the specifications.

The mix called for is generally one part cement, two sand, and four of gravel or stone, or a mixture of both gravel and stone. A leaner mix is sometimes permitted for the protective course of concrete outside the waterproofing, where waterproofing is employed. The concrete surrounding the steel shells of the new tunnel under the Harlem River, described later on, is specified as a 1 : 3 : 6 mix.

Within limits, and subject to the inspection of the Commission, the methods of mixing and placing are left to the discretion of the several contractors, and, as might be expected, this has led to a lack of uniformity of method. In the case of many of the contracts, notably most of those for the Lexington Avenue sections, the contractors preferred to use central mixing plants, which were generally located along the East River where the material could be brought to them by water, the concrete being hauled, after mixing, one-half mile or so to the work. Where motor trucks were used, the time consumed in transporting the mixture was not a cause for concern; some of these trucks had a capacity of 4 cu. yd. On one of the sections air jets were used instead of water jets to assist in loosening the concrete from the truck when being dumped. On several of the contracts, the concrete was hauled in horse-drawn trucks from the mixing plant, and the time required was long enough to raise the question whether initial set was taking place before the concrete could be placed in the forms, and it was necessary at times to reject some of the concrete where it was delayed too long in transit. Because of the trouble from this source, the contractor then adopted the plan of mixing the ingredients dry at the central plant and delivering the dry mixture on the street at the work, when water was added and the mixture turned over twice by hand before being placed in the forms.

Among the advantages of a central mixing plant are the following: Economy, especially when it is located on the water front; a minimum of inconvenience to the public and to residents along the line of the work; and more convenient supervision of the whole operation of mixing. In spite of these advantages, however, the author prefers to have the concrete mixed as close to the work as possible, as he believes that a better quality of concrete is insured when it can be delivered to the forms immediately after mixing. On those contracts where the latter method is followed, portable mixers are generally used, being moved from place to place as the progress of the work demands. This, of course, means that the street must be temporarily obstructed, not only by the mixing plant itself, but by the piles of concrete

material which must be stored in the vicinity, and has been the cause of many complaints by owners and tenants of adjacent property. As most of the streets are decked over the subway openings, the concrete is delivered to the forms through chutes leading from holes cut through the decking, and relatively little of the concrete is mixed in the cuts below the street surface.

Mixing and placing concrete by a pneumatic method has been tried experimentally on several of the subway contracts, principally on Section 5 of the Broadway-Fourth Avenue Subway, where an extensive trial has been made of placing the concrete lining of the tunnels, as well as that for the structure in open cut. The following description prepared by Mr. A. E. Comstock, the Assistant Engineer in direct charge of the work, will be of interest:

"On Section 5 of the Broadway-Fourth Avenue subway, which includes the two-track structure on Fifty-ninth and Sixtieth Streets from Seventh Avenue east to Second Avenue, the concrete lining of the tunnels and other structural concrete are both being placed by compressed air. The machine used for this purpose is designed for both mixing and placing concrete. It consists of a drum about 4 ft. in diameter by 2 ft. deep, on the underside of which is attached an inverted cone-shaped section extending about 3 ft. below the bottom of the drum and terminating in a 6-in. discharge pipe. On top of the drum is a feeding hopper, and inside is a horizontal door, sliding in cams so adjusted that in closing it is brought tightly against a gasket on the top of the drum, thus making an air-tight lock. The door is operated by a hand lever connected to it by a piston rod passing through the drum. The cone-shaped section underneath the drum is fitted near its top and bottom with about twelve small inlet pipes through which air is forced for the purpose of agitating the materials until they are thoroughly mixed before the mixture is driven through the discharge pipe to the forms. The discharge pipe is attached to the cone by a T-connection, from the free end of which a 2-in. pipe leads to the compressed air reservoir and another 2-in. air connection is located at the top of the drum. When the process of mixing has been completed, the air is turned on at both connections; the air through the upper one forces the concrete down into the discharge pipe while the air through the lower one forces it forward to the forms.

"When the machine was first put in operation on this contract the agitators were not used, as it was thought that the materials would become sufficiently mixed in transit through the discharge pipe and upon being deposited in the forms. Repeated trials made by this method proved unsuccessful, the aggregates failing to mix properly with the cement and water and were deposited in the forms in a condition varying from very wet to dry, and being deposited under an initial pressure close to 100 lb., the materials that had become only slightly wet were compacted to such an extent that rehandling was difficult and involved considerable loss of time, and furthermore an appreciable amount of sand and cement that had not come in contact with the water was blown clear of the forms. Later developments showed that dry materials had a tendency to clog the pipe more frequently than when thoroughly mixed with water. Various methods and contrivances were used

to overcome these difficulties and obtain a better mix, but without satisfactory results. An attempt was then made to use the agitators, but these were found to have become obstructed with cement to such an extent that it would have required a considerable loss of time to clear them; moreover it was the opinion of the engineers that they would fail to do the work for which they were designed, for when the materials are placed in the mixer they drop first into the discharge pipe and fill upward from that point, and all materials below the lower set of agitators would not be affected by them. Also the agitators would act simply as an air inlet and would immediately cause the discharge of the materials from the machine before mixing could take place.

"An arrangement was then made by which the materials could be thoroughly mixed to the proper consistency before being placed in the air mixer. This method with proper spading and sufficient rehandling in the forms when necessary to insure an even distribution of materials has resulted satisfactorily and is the method now being used. The arrangement of the plant at the mixing point was such that with a slight modification this method of mixing could be used continuously. Two storage bins, one for sand and one for gravel, each with a capacity of about 12 cu. yd., were erected with their tops at the level of the street in such a position that trucks discharge into them conveniently. At the bottom of each bin is a gate through which the materials are run into a hopper gaged for measuring the sand and gravel. At this point there is a platform, covered and waterproofed, where the cement is handled and temporarily stored, on which is located a water tank fitted with an improvised measuring gage and connected to a steam boiler for the purpose of heating in cold weather. The steam pipes also pass through the storage bins for the same purpose. At the bottom of the measuring hopper is one gate which, when opened, allows the sand and gravel, to which the cement has been added, to run together into the mixer. Before experimenting with the air mixer a rotating mixer had been in use in placing the track floor and wall benches. Space was provided in the arrangement of the plant to set aside one mixer in case it was desired to use another. Upon the failure of the air machine to properly mix, it was replaced by the rotating machine, lowered to a position above the air machine and so arranged that the rotating machine could thoroughly mix the materials and discharge into the latter with the above mentioned satisfactory results.

"The discharge pipe should be laid in the most direct line between the mixing point and the forms, avoiding bends as much as possible, since all bends retard the passage of concrete through the pipe, encourage congestion, and increase the time of transit as well as the consumption of compressed air. Where bends are necessary they should be sweeps of large radii, about 15 or 20 ft. Short bends of 4 or 5 ft. were used on this contract in the early experiments with this method, and were found to clog more frequently than those of larger radii and to wear through in a few hours, with consequent loss of time for repairs. When the pipe becomes clogged, sounding at the point of congestion will usually loosen the obstruction, but it is sometimes necessary to remove the obstructed part and clear it by other means. At every point where the pipe changes direction there is a decided shock when the concrete

strikes the bend, and it is therefore necessary to have such points securely braced in order to prevent unnecessary strain on the joints, particularly at the outlet, where the shock would otherwise cause a shaking and consequent loosening and displacement of the forms, where there is any contact between the pipe and the forms, a condition which cannot always be avoided. The original position of the outlet of the discharge pipe should be located centrally in the forms, since each change of pipe means additional loss of time for bracing. In passing through the discharge pipe, especially over long distances, the concrete has a tendency to spread out along the bottom rather than move in a solid mass through the full section of the pipe. If this condition continued to the outlet of the pipe much of the air would pass over the materials, resulting in a loss of pressure, a longer time in discharging, and a separation of materials at the outlet. In order to avoid this it is necessary to have a rise, vertical or nearly so, at or near the point of discharge so as to cause the concrete to more completely fill the pipe and to discharge more uniformly.

"In order to obtain the proper distribution of concrete in the forms with as little rehandling as possible, a flexible rubber pipe was attached at the outlet with the idea of directing the flow to different points, as desired. This proved practically useless, as the pipe buckled, was difficult to control, and was soon worn out; it was therefore replaced by a T-connection which could be turned to any desired angle in a vertical plane, and satisfactorily provided the required distribution. This method of placing concrete is particularly adapted to filling tunnel arches, the discharge pipe being laid along the crown of the arch to about the middle point of length of the section, and the T turned alternately on either side until the arch is filled to the level of the crown. The T is then removed and replaced by a piece of straight pipe extending to within a few feet of the far end of the form, and the whole length of pipe inside the form is raised to a position tight against the rock along the highest available line, and concrete is pumped in until the arch is filled up to the outlet of the pipe. The pipe is then removed and replaced by a shorter piece extending about 6 in. into the form at the most advantageous high point, the bulkhead is sealed and the blowing of concrete resumed until the arch is filled to the point of refusal. The pipe is left in place until the concrete has set long enough to be firm, when it is removed and cleared of any concrete remaining in it. It is impossible to completely seal all voids in the roof of a tunnel by this method of placing concrete unless the rock breaks fairly smooth without leaving pockets above the general surface.

"Care must be taken to avoid overtaxing the forms, as the high-pressure of air added to the weight of the concrete is likely to displace or rupture them, and such an accident actually occurred at one point on this work. On this contract the point of deposit farthest from the mixer is about 500 ft., and the time required to discharge a two-bag batch of concrete over this distance varies from 30 to 40 seconds. Each batch is mixed for about one minute, during part of which time the previous batch is discharged into the forms. Under favorable conditions about $1\frac{1}{2}$ minutes are required to mix and place each batch, but on account of time lost in spading the mix in the forms, occasionally changing the position of the pipe, clearing obstructions, and waiting

for the proper amount of initial air pressure, a fair average between batches may be assumed as about 3 minutes. The time required decreases in length as the point of deposit approaches nearer to the mixer. It therefore requires about 12 hours to line a section of tunnel roof 25 ft. long which contains about 80 cu. yd. of concrete, including excess. At the present time there are two single tunnels and a double tunnel, each about 300 ft. in length, being constructed by this process. Part of this double tunnel is open cut construction where complete tunnel sections above track floor, each 10 ft. long, are being poured at one time as monoliths. The work is in progress at three points, thus keeping the machine in practically constant use. Steel forms and carriers are being used for tunnel sidewalls, and timber arch forms on steel carriers for the tunnel roof."

Fig. 1 is a view of concrete placed by this method.

PRECAUTIONS AGAINST FREEZING.

As by far the greater part of the subway work is being carried on under decking, or in tunnel, no special precautions are necessary to protect the fresh concrete from the hot summer sun, and in freezing weather the aggregates are heated to remove the frost. The usual means of doing this is to pile the material over large pipes lying flat in the street, in which wood fires are maintained. The water is also heated and the fresh concrete in place is protected by a covering of salt, hay, canvas or other acceptable material. A method employed for heating the concrete aggregates during cold weather is described in part by Andrew Veitch, Assistant Division Engineer of the Commission, as follows:

"The method now in use by the contractors on Seventh Avenue, Routes 4 and 38, Sections 5 and 6, United States Realty & Improvement Company and Rapid Transit Subway Construction Company, is an attachment placed on the mixer which forces a flame or hot blast directly into the mixer, thus raising the mix to the required temperature. Almost any degree of heat may be obtained depending on the length of time the materials are left in the mixer. If the heat should tend to dry out the mix, sufficient water must be added to maintain the right consistency for use. The heating apparatus consists of the ordinary kerosene blow torch inserted in a large pipe placed at the hopper through which the heat is forced and directed into the mixer. From the 30-gal. kerosene tank placed on top of the mixer the oil feeds into the burner where it is vaporized and burns, and the flame is forced into the mixer by compressed air. This compressed air is supplied by a direct connection to contractor's air line. It is evident that by using this method there is a considerable saving to the contractor in labor of handling the materials; likewise much saving of space on the streets in cases where materials can be used directly as carted from the docks or store yards."

When it is considered necessary, a tarpaulin is hung at each end of a section of forms, and a salamander using coke is kept burning as long as necessary to keep the temperature above the freezing point. It should be

noted that more protection is needed where steel forms are used than when wooden ones are employed, steel being a better conductor than wood. It is the opinion of the author that concreting for subway work can safely be

FIG. 1.—VIEW OF CONCRETE ON SECTION 5 OF ROUTE 4 AND 36, FIFTY-NINTH STREET, MANHATTAN.

This is an example of the concrete mixed in a rotary batch mixer and transported to the form by pneumatic methods.

carried on regardless of the ordinary low temperatures of New York City, if unfrozen material is used in the mixture, and if the fresh concrete is properly protected against freezing until it sets.

FORMS.

The contracts specify that "the forms shall be made of wood, kept carefully planed; or made of metal sufficiently thick to retain their shape without the use of wood. . . . No forms made of wood and covered with iron will be permitted. . . . The forms, if made of wood, shall be made of boards with tight joints, tongued and grooved, if required by the engineer." It is left to the choice of the contractor whether he should use wood or steel forms. On many of the contracts the contractor prefers wooden forms, particularly for much of the special work, particularly that about the stations, ventilating chambers, etc., where the forms can only be used once on account of the special character of the work.

PLACING EXTERIOR CONCRETE FOR THE HARLEM RIVER TUBES.

The Harlem River Section of the Lexington Avenue Subway, known as Section 14 of Route 5, was built by sinking steel tubes in sections in a trench

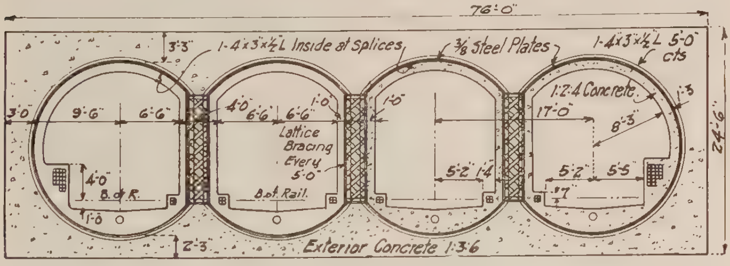

FIG. 2.—CROSS SECTION OF HARLEM RIVER TUNNEL OF THE NEW LEXINGTON AVENUE SUBWAY.

previously dredged in the bottom of the river. The steel shells were then surrounded with concrete placed under water through tremies, and the work was carried on in a manner generally similar to that employed in the construction of the Detroit River tunnel. The Arthur McMullen & Hoff Company was the contractor; as is generally known, Mr. Olaf Hoff of the company was connected with the construction of the Detroit River tunnel.

The total length of the steel tunnel is 1080 ft. and it consists of four sections, *A*, *B*, *C* and *D*, each 220 ft. long and one section, *E*, 200 ft. long. The lowest point of the main structure is 57 ft. below mean high water. Fig. 2 shows a cross section of the tunnel and indicates four parallel tubes 17 ft. apart on centers, having internal diameters of 19 ft. with flat sides on their interior walls. The concrete structure surrounding the tubes is 76 ft. wide by 24 ft. 6 in. high. At intervals of 15 ft. 7 in. are vertical steel diaphragms at right angles to the axis of the tunnel, consisting of ¼-in. plates stiffened by angles. To the vertical faces of these diaphragms were attached horizontal 4 x 12-in. lagging, thus forming pockets 76 ft. wide by 15 ft. 7 in. long, as indicated in Figs. 3 and 4. The interesting methods employed in the erec-

tion, launching, and sinking of the sections of this tunnel are well described in a paper prepared by Mr. Howard B. Gates, Assistant Engineer of the Commission, for the Municipal Engineers of the City of New York and presented at their meeting on December 22, 1915. His description of the placing of the outside concrete follows:

"With the section in place, the encasing of it was started generally the next day. Briefly, the special plant employed consisted of three complete mixing plants serving five tremie pipes mounted on wooden towers, with two large boilers to furnish power for the operation of the various engines and hoists, including the loading derrick. The wooden towers, 50 ft. in height and

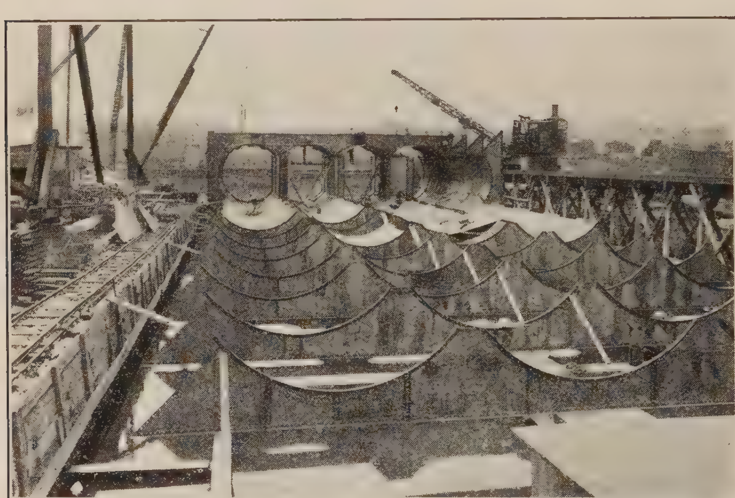

FIG. 3.—ASSEMBLING THE STEEL WORK FOR ONE OF THE SECTIONS OF THE NEW HARLEM RIVER TUNNEL FOR THE LEXINGTON AVENUE SUBWAY.

6 ft. 6 in. square, were spaced about 17 ft. on centers and served the double purpose of a guide for the tremie-charging bucket and for supporting the tremie hopper to which the tremie pipe was attached. The charging buckets were filled by three rotary batch mixers, one mixer serving each pair of extreme pipes and one the center pipe. Aggregate bins holding about 20 cu. yd. each of sand and gravel were placed directly above the mixers which they served and were supplied by means of the derrick and clam-shell bucket, from barges tied alongside. Although there was provided a storage room large enough to hold about 250 bbl., the cement was usually obtained directly from the cement barge in the same manner as the aggregate. An old freight car lighter, 37 ft. wide, was cut down so that it was 140 ft. long and served as a float upon which this plant was assembled. (Figs. 5 and 6.) Fresh water

was required in the mixing of concrete so that water-storage tanks were built in the bottom portion of the barge.

"The tremies were made of 12-in. diameter spiral riveted pipe 63 ft. long, suspended from a movable hopper which was attached to the front legs of the tower and could be raised or lowered at the will of the tremie operator from the hopper platform by means of an endless rope, working over a series of pulleys which controlled the throttle of a single-drum hoisting engine. A ball-and-socket swivel joint was placed about 30 ft. below the hopper in order to accommodate any shifting of the scow or listing due to the operation of the loading derrick or swells from passing boats. This provision gave the

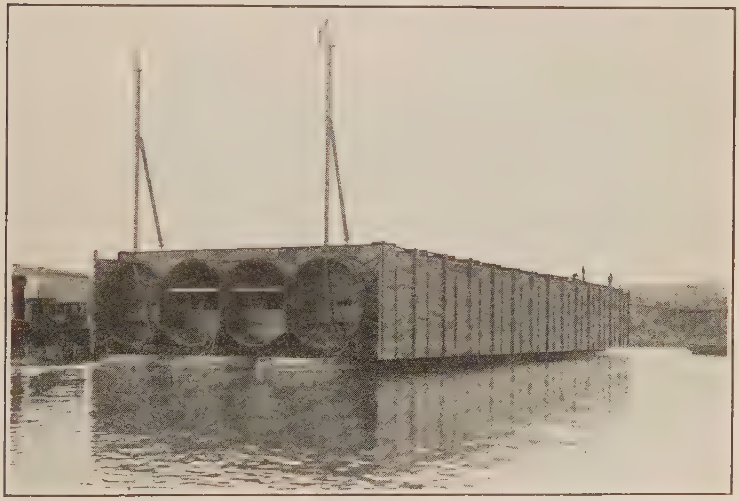

FIG. 4.—SECTION OF THE STEEL TUBES FOR THE NEW HARLEM RIVER TUNNEL OF THE LEXINGTON AVENUE SUBWAY AFTER LAUNCHING, READY TO BE TOWED TO THE SITE OF THE TUNNEL AND SUNK IN PLACE.

pipe ample flexibility and without doubt saved many annoying delays from broken pipes and lost batches of concrete. The charging buckets for the tremies were operated inside of the square towers and were arranged by a tripping device to discharge automatically into the tremie hoppers.

"In their operation the tremies were first guided into position by the diver, a pipe being placed on each side of the four tubes, and one dry batch of concrete was dumped into each of the pipes, acting as a plunger to force the water out and seal the bottom, immediately after which they were filled with wet concrete, two batches being sufficient to fill the pipe. As fast as the concrete oozed or boiled out at the bottom of the tremies, it was supplied at the top. The essential requirement for the satisfactory operation of the

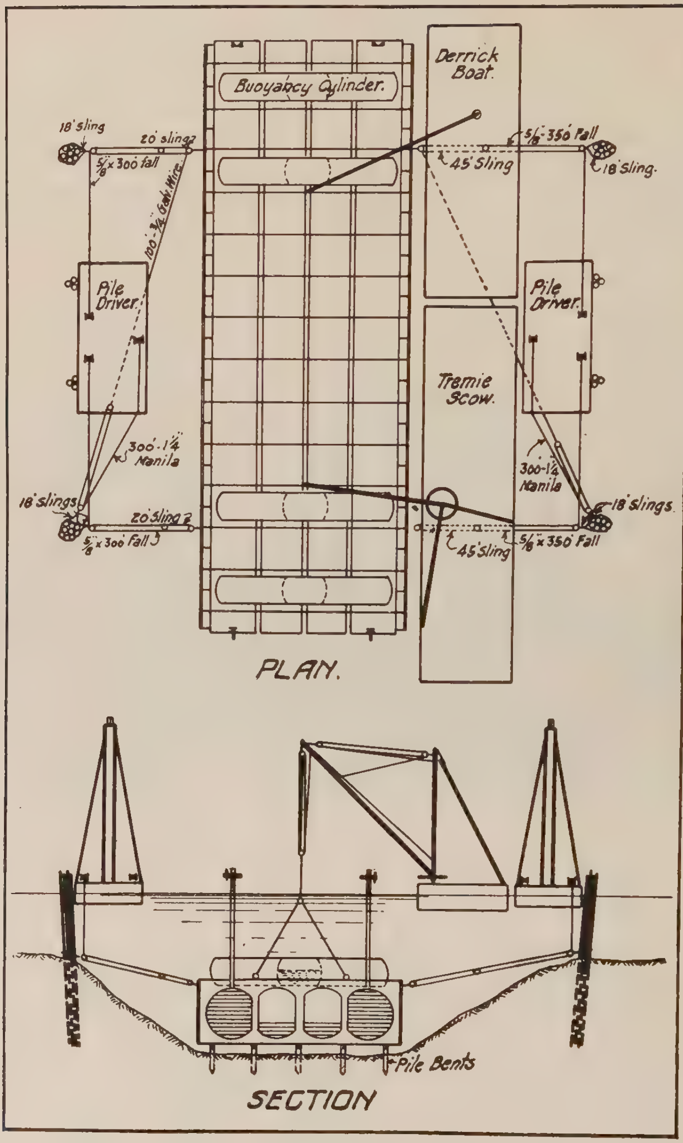

FIG. 5.—TYPICAL PLAN FOR LOWERING OF STEEL TUBES FOR THE NEW HARLEM RIVER TUNNEL OF THE LEXINGTON AVENUE SUBWAY.

This view shows the tremie scow assisting in the lowering of the tunnels. While the concrete is in progress, the position of this scow is right angle to the axis of the tunnel. This scow is floated into position across the tunnel and moved longitudinally. The concrete is placed in each pocket between the diaphragms.

tremie method of depositing is to maintain a continuous or nearly continuous flow of concrete directly to its final position with as little movement as possible and gradually replace the water with concrete but without any mixing of them except at the contact surfaces. In meeting this requirement the tremie pipes were kept full of concrete, maintaining a practically constant head upon the discharge end, and by keeping the end of the pipe submerged from 3 to 6 ft. in the soft concrete, the flow was readily controlled by raising or lowering the pipe. The tremie operators became so expert in controlling the discharge that very seldom was any trouble experienced with the sudden emptying of the pipe and the subsequent filling of the tremie with water.

"At the beginning of concreting, the structure might be considered as a large box with sides and ends and a series of partitions at regular intervals but without a top or bottom. To form the bottom of the box and to pro-

FIG. 6.—FRONT VIEW OF TREMIE SCOW USED FOR PLACING THE EXTERIOR CONCRETE FOR THE NEW HARLEM RIVER TUNNEL OF THE LEXINGTON AVENUE SUBWAY.

vide a bearing for the diaphragms, concrete of the proportions 1 : 4 : 8 was deposited with the tremie and on the average was 3.5 ft. in thickness, representing the amount of the overdredging. The use of a mixture of sand and gravel or any compacted materials would have met the contract requirements in the replacing of this over-dredged volume, but on account of the difficulties of placing, it is doubtful if there would have been any economy effected over the use of this lean mixture of concrete. With the exception of Section E, practically all of this foundation concrete was deposited before any of the enveloping concrete was placed, except one or two pockets near the free end to assure a positive anchorage. The pockets were then filled in any convenient order until the entire section was encased, two foundations or one pocket being usually considered a day's work, although two pockets were finished occasionally when an early start was possible. The contractor was

naturally required to carry on his operations in any pocket continuously to its completion. In the case of Section E, where it was decided to complete each pocket, including the foundation, working continuously, it is now thought that this procedure gave a more uniform contact between the concrete and the steel work and prevented any accumulation of laitance or deposition of any kind upon the surface of the foundation concrete from becoming trapped and incorporated in the encasing concrete. An inspection was always made by a diver of the surface of the foundation concrete and any amount

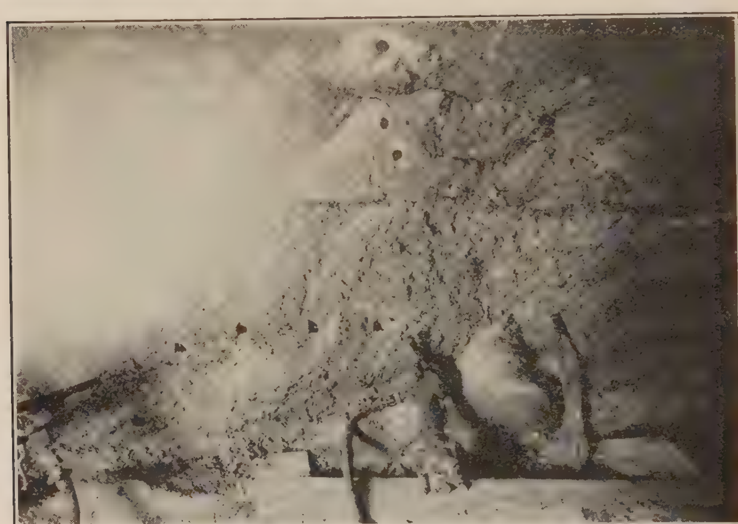

FIG. 7.—CUTTING THROUGH THE CONCRETE FOUNDATION OF THE BULKHEAD WALL AT OLD SLIP, MANHATTAN, FOR THE NEW OLD SLIP-CLARK STREET TUNNEL UNDER THE EAST RIVER, NOW BEING DRIVEN BY COMPRESSED AIR.

This concrete was deposited on the river bottom in burlap bags by the Department of Docks and Ferries, New York City, in 1905. At the top of the view will be seen the load of the tunnel shield.

of foreign material removed before filling the pockets. It required on the average 22 working days of 8 hours to place the exterior concrete for each of the 220-ft. sections. The average output of the three mixers working eight hours was 360 cu. yd. with a maximum of 700 cu. yd. The plant was operated by a crew of 40 men, including the diver and his helpers."

CONDITION OF OLD CONCRETE.

The original Interborough subways, known as Contracts 1 and 2, which are now in operation, were constructed between 1900 and 1906. It has been

necessary from time to time to lengthen the platforms of the stations to provide for longer trains, to construct ventilation chambers, emergency exits, and to make other alterations in the structure for various purposes, such as the connections at different points with the new subways. All of this work involved the cutting out of the original concrete in many places, and it is a satisfaction to state that there has been no evidence of any disintegration of the old concrete from electrolysis or other causes. Two pieces of concrete from the lining of the Park Avenue tunnel were cut out within the past few months, sawed into 6 x 6 x 12 in. prisms and tested for compression in the physical laboratory. One sample failed at 3950 lb. per sq. in.,

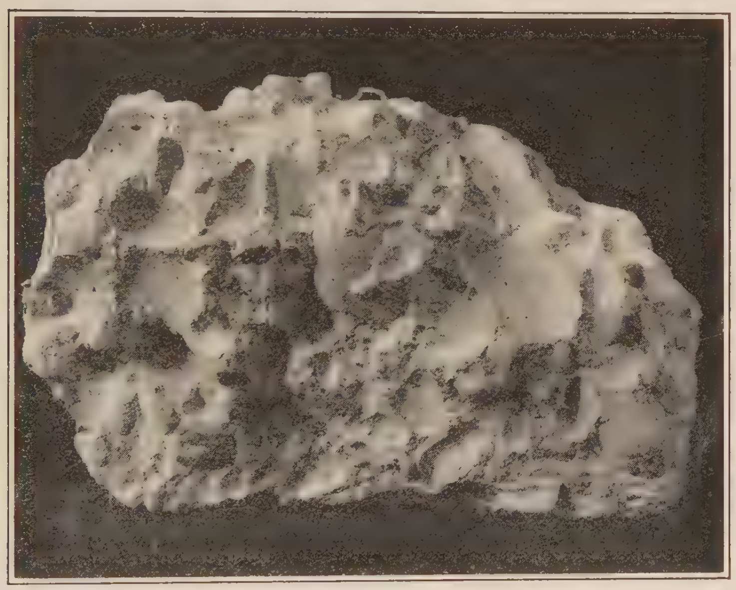

FIG. 8.—SAMPLE OF CONCRETE FROM FOUNDATION OF BULKHEAD WALL AT FOOT OF OLD SLIP, MANHATTAN, SHOWN IN FIG. 7.

the other did not fail at the capacity of the machine, about 4000 lb. per sq. in. The concrete was 13 years old and the samples were selected by the Assistant Engineer as being rather below the average quality of the concrete which was being removed in the vicinity.

In driving the shields for the tunnel which passes under the East River from Old Slip, Manhattan, to Clark Street, Brooklyn, the Manhattan shields encountered the concrete foundation of a bulkhead wall which was constructed by the Department of Docks and Ferries in 1905. This foundation was in the form of concrete deposited in the salt water in burlap bags, and was found to be in excellent condition, as Figs. 7 and 8 show. The burlap of the bags

was intact in places, and the general appearance of the material was that of mass concrete placed in the dry.

Mr. Ralph E. Goodwin, Junior Engineer of the Commission, who was to have prepared this paper, was, unfortunately, unable to undertake the task on account of illness, and the author regrets that the demands on his own time have been such that he has been obliged to put together the facts in a rather hasty way and feels that he has not done full justice to the subject. He wishes to express his indebtedness to the members of the Engineering Department of the Commission, whose names are referred to in this paper, and to Mr. George L. Lucas, General Inspector of Materials, and his assistants for their help. The work described is under the direction of Mr. Alfred Craven, Chief Engineer of the Commission.

RELINING A TUNNEL WITH STEAM-JETTED CONCRETE.

By Harold P. Brown.*

For many years a great deal of trouble has been caused to the engineers of the Chicago Great Western Railroad by the single track tunnel 2600 ft. long at Winston, Ill. This is driven through gray shale which disintegrates into fine clay on exposure to air. The overlying strata are full of water and the tunnel has always been very wet. When first built it was timber-lined, and in 1886 a portion collapsed, blocking traffic for many months. A brick lining was then placed inside the timber and the contractors were supposed to fill the space between the two. A $1\frac{1}{2}$-ft. grade is encountered through the tunnel and the east-bound freight traffic frequently requires double headers.

A ventilating shaft was driven in the center of the tunnel when it was relined, but as this failed to relieve the smoke nuisance, a fan run by a Diesel oil engine was placed at the west end to clear the smoke ahead of east-bound trains. But the combination of smoke, water and freezing during the winter, due to the use of the fan, caused a serious disintegration of the brick, especially on the arch. Although analysis showed that the water was very pure, wherever the exhaust blast encountered the wet roof, the bricks laminated into strata about $\frac{1}{16}$ in. thick, parallel with one face, sometimes the base, sometimes the side and sometimes the end. Between the layers was a black, gummy substance having an acid reaction. Nearly all the lower layer of the arch had fallen away, quite a portion of the second layer was cracked and a little of the third. Although a number of small weep holes were provided, they quickly became filled up with fine clay and were ineffective, leaving the brick saturated with water, which came down in streams, especially at the eastern end. Several thousand dollars were expended each winter in removing ice from the tunnel, and the relining was a serious problem. There was not sufficient clearance inside of the old lining to permit placing a new one strong enough to sustain the load. To take out the old lining and put a new one in its place while maintaining service through the tunnel was too dangerous to consider. The track levels at the portals and the treacherous character of the ground beneath the foundation prevented a lowering of the tunnel grade to accommodate a new roof lining during maintenance of service.

About a year ago the writer was called upon to examine the tunnel and suggest a method of relining which would meet the very difficult and unusual conditions. It was evident that the foundation was in satisfactory shape and that the side walls were but little injured. The roof, however, was in dangerous condition and required a lining which would take its proper share of the load.

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* New York, N. Y.

My report advised a slight lowering of the track level; the drilling of a large number of weep holes 3 in. in diameter; the washing out of the clay between the upper brick and the old timber lining and filling the space with grout under pressure, and the removal of the cracked brick. These should at once be replaced by an adhering layer of steam-jetted concrete continued 2 in. below the old surface and sufficiently reinforced, if necessary, to take its share of the load.

In August, 1915, a work train was equipped for the job. The engine was provided with an extra air compressor, a steam pump and a dynamo for electric lighting. A pressure-reducing valve set at 90 lb. was connected from an extra heavy nipple on the dome and a 2-in. steam connection carried with Franklin ball-and-socket joints and suitable couplings to a flat car on

FIG. 1.—THE CONSTRUCTION TRAIN FOR RELINING WINSTON TUNNEL. NOZZLE CAR AND PART OF CLEANING CAR IN FOREGROUND.

which was placed the concrete atomizer. The same car carried the cement, sand and gravel.

As it would be difficult to control by hand a nozzle for jetting the concrete on to the roof, I designed for this purpose a nozzle car and troweling machine which would place the concrete, would indicate the depth applied and would trowel or finish the final layer. The second flat car in the train carried this machine, which was mounted on a platform capable of vertical or lateral adjustment, so that it could be made to swing from the center line of the arch. The nozzle was secured to a shaft mounted on suitable journals and was moved from side to side by a reversible two-cylinder steam engine. The same shaft carried the distance indicator and the troweling devices. The nozzle car was mounted on wheels running on channel irons and could

be moved back and forth 10 ft. by means of a stationary windlass. Steam connections to the engine and to the nozzle were made by means of suspended lengths of wire-protected rubber hose.

Beyond the second flat car was a box car provided with a railed platform on the roof. Here three men operated a water jet to clean the soot

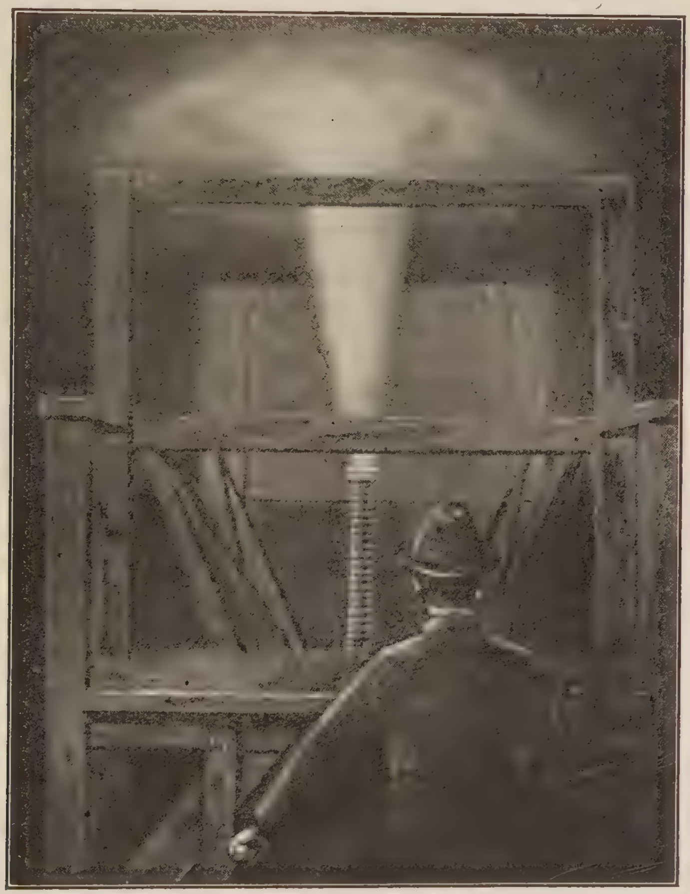

FIG. 2.—PLACING STEAM-JETTED CONCRETE ON TUNNEL ROOF.

and dirt from the walls, and light pneumatic hammers for removing the defective brick. A signal whistle mounted on the engine, which could be sounded from any part of the train, was used to control the flow of air, water, steam and concrete. Two acetylene headlights as well as a number of incandescent lamps were used for general illumination.

When the work was started on September 4th, I found that the pneumatic drills provided were not heavy enough to penetrate the brickwork for the necessary 3-in. weep holes. Rather than wait until the proper drills could be obtained, I started operations at the eastern portal where water was pouring in streams through the roof. A mixture of one part cement, three parts of sand and two parts of pebbles was used with 10 per cent of water. These were mixed in the concrete atomizer at 90 lb. pressure, with the superheat obtained by dropping from the engine working pressure

FIG. 3.—CONCRETE ATOMIZER AND 20 H. P. BOILER ON LACKAWANNA RAILROAD WORK.

through a reducing valve. The concrete was carried through 2-in. hose and shot on to the brick by a steam jet. Although the pebbles at first dropped away, they nevertheless forced the mortar into all the interstices of the brickwork and checked the flow of water. But very little material bounded off and this was collected and used.

Before shooting a load, steam was jetted through the nozzle to complete the cleaning of the brickwork and heat it to the temperature of the concrete. In some places a layer of concrete 7 in. thick was jetted on to the

roof and it set up so quickly that the work train could be immediately followed by an east-bound freight without trouble resulting from the engine exhaust.

In the final layer lime hydrate was added and the pebbles were omitted. The proportions were one part of hydrate to ten parts of cement and thirty parts of sand. This gave a smooth-flowing mixture and delayed the setting so that when the steam jet from the nozzle car was passed over the surface a second time without concrete, a smooth finish was obtained and troweling was unnecessary.

The work was in charge of my superintendent, while the men composing the crew were employees of the road. Three men cleaned the roof, one

FIG. 4.—INTERIOR OF HYDRAULIC TUNNEL NEAR RUPERT, PA., BEFORE LINING WITH STEAM-JETTED CONCRETE.

man operated the nozzle car, one worked the windlass, one ran the atomizer and two men measured and loaded the materials. The improved machine is arranged so that but one man is needed to work the nozzle car and the windlass. It required about 5 minutes for the gang to clean 10 ft. of roof, load the atomizer, steam the roof and treat and apply the load. As it was important not to interfere with train service upon a double-track road through a single-track tunnel, only about 6 working hours per day were available in the tunnel. An average of 262 ft. of lining 12 ft. wide and 3 to 4 in. thick was applied in 6 hours, using 35 bags of cement. The concrete was found to be so strong that no reinforcement was needed, nor was it necessary to fill with grout the space above the arch. The work was finished in 41 days,

its total cost was $4600, and Mr. C. G. Delo, chief engineer, has pronounced it entirely satisfactory.

It was found that there was but little wear of the rubber hose when pure Para gum was used, but if inferior rubber were substituted the swift stream of hot concrete soon cut through the fabric. This led to the invention of an interior armor for ordinary iron pipe which can not be injured by concrete. This pipe is also covered with magnesia insulation which is protected from mechanical injury by a spiral rivetted pipe 5 in. in diameter, made of No. 26 galvanized sheet steel. The sections are 15 ft. long and stand 3 in. above the ground on wrought iron chairs. I use quick-detachable

FIG. 5.—INTERIOR OF HYDRAULIC TUNNEL NEAR RUPERT, PA., AFTER LINING WITH STEAM-JETTED CONCRETE.

couplings which require no wrenches nor gaskets. These permit 15 deg. variation of line between two sections. For curves, short lengths of flexible metal pipe are used and these also have the protecting armor.

I have thus overcome the greatest drawback to the use of steam for moving and jetting concrete, and the rate of delivery is greatly increased by the mechanical control of the nozzle and the troweling devices. Twenty cu. ft. can now be discharged in 30 seconds, while this required 3 or 4 minutes with a man at the nozzle. This has led to the design of other nozzle cars for jetting circular grain bins, silos or stacks with thin walls of dense reinforced concrete. The way is now open for the construction of larger machines to mix concrete with 2-in. stone and convey it several hundred feet through a 3-in. pipe.

DISCUSSION.

After the presentation of the paper the author showed a number of lantern slides and answered numerous questions. The information thus given may be summed up as follows:

Mr. Brown.

Mr. Brown.—In the Winston tunnel after the relining was completed the weep holes were kept open in order to drain away the water and thus reduce the pressure on the concrete lining. A conduit can be made water-tight without weep holes by the use of a thin layer of steam-jetted concrete.

There is very little loss of material by rebounding; all the pebbles thrown against the wall to form the first or inner part of the lining bound away but are recovered and used a second time. It is not possible to completely fill a small depression or cavity with a mixture of sand and cement only, because little pockets of air intervene between the mortar and the old concrete or brickwork. But when small pebbles are added to the mortar they seem to act as hammers to drive the mortar into all cavities and secure perfect adhesion.

Although hydrated lime was used in the final layer of the tunnel lining and undoubtedly helped in making the thin lining waterproof, the density of the material applied with steam is so great that waterproofing compounds are unnecessary. The amount of hydrated lime was 10 per cent of the sand and cement, by weight. The sand is first put in the atomizer, then the cement and hydrated lime and then the water and aggregate. All are measured out each time. Steam-jetted concrete sets almost instantly, unless hydrated lime is used with it to delay the setting and permit troweling. In no work of this class that has been examined has there been any evidence of unhydrated cement. The density of the steam-jetted concrete is shown by its weight, 171 lb. as compared with 137 lb. per cu. ft. of concrete made of the same materials in the same proportions cast and hand tamped.

The water added amounts to 10 to 12 per cent of the weight of the sand and cement. In addition about 4 to 6 per cent of water is probably added by condensation of the steam. Too much water prevents the adhesion of the new concrete to the old surface when it is thrown overhead. If saturated steam at a pressure of 60 lb. is turned on cold materials the condensation adds too much water. To avoid this the steam supply pipe is given one or two turns in the stack. This adds sufficient superheat to compensate for loss in transmission through several hundred feet of pipe or hose; it also aids crystallization and secures greater density in the concrete.

In the Winston tunnel the nozzle was held about 4 ft. from the wall, but where a thin smooth layer is to be put on, the distance of the nozzle from the work is increased to 8 ft.

Trials of steam-jetted concrete on pavements have been made success-

Mr. Brown. fully. For this purpose the Concrete Atomizer is mounted on wheels, as is the boiler which supplies it with superheated steam. It has been found necessary to remove every trace of asphalt or tar from the surface of the pavement, as the presence of either will prevent the steam-jetted concrete from adhering to the old concrete, stone or brick. It can be made to adhere to old cobble stones if they are well cleaned and wet. The nozzle should be held not more than 3 ft. away and the pressure should be 50 to 60 lb. For concrete roads a gooseneck is mounted between a pair of wheels in such a way that the material is shot directly down and the man controlling the operation can see what he is doing and avoid the rebound of the pebbles.

REINFORCED CONCRETE IN SEWER CONSTRUCTION.

By W. W. Horner.*

Up to 1900 sewers were constructed of brick and stone, sometimes accompanied by concrete backing and occasionally of plain concrete. From long experience with these materials, certain typical sections had been developed, of which the circular and egg-shaped brick sewers in the smaller sizes were more common. For larger sizes some form of arch with a generally flat invert was used. In much of the older work the base or bottom was actually a timber grillage, and contained lumber up to 12 x 30-in. sticks. In the later construction an inverted arch of cut stone or brick on a concrete bed, was substituted for the timber. Above the base the sewer consisted of heavy sidewalls or abutments and a semi-circular brick or stone arch, haunched with rubble masonry and later with concrete. Occasionally on projects of great magnitude, a closer study of the lines of pressure resulted in modifiactions of this type, and some examples of elliptical, catenary and parabolic arches were found. It appears evident that most of the sewer arches in that period were not the result of stress analysis but were proportioned by arbitrary rules resulting from a study of existing structures.

About 1900 reinforced concrete had been taken up in many fields of construction and had been applied to long-span arches, and the advantages of its use in sewer construction were soon appreciated. During the next five years, many examples of concrete sewers are found, although with a few exceptions the reinforcement consisted of wire mesh or expanded metal, and there was an evident tendency on the part of the majority of engineers to use rather heavy sections similar to the plain masonry types. A few examples are also found of extremely radical designs involving very light sections heavily reinforced. These two extremes suggest the difference between the sewer engineer adapting his designs to reinforced work and the concrete expert breaking into the sewer field. In the last ten years, all of these ideas have been through the melting pot, and we are beginning to find certain standard types of reinforced concrete sewers used generally. These are the horseshoe type varying in proportions from the semi-circular to those of about equal height and width, and the elliptical usually constructed as a five-centered arch. Of exceptional advantage under certain conditions the box or slab section is often employed, but under average conditions it is less economical than the other types. The circular sewer is difficult to construct in what is known as "monolithic" work, that is, if built in place, but the circular reinforced concrete pipe developed along other lines has become standard construction in size up to about 8 ft. It is unit work and may be considered as a factory product, and for that reason a much more satisfactory concrete can be secured through its use than is generally obtained in monolithic work.

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CROSS-SECTIONS.

In many cases, the shape of the sewer will be controlled by local conditions. In wet ground the invert must be kept as high as possible and a broad shallow section results. For such cases the semi-circular shape is the most economical, and in fact is about the limit of distortion in that direction, as computations show that little further decrease in height can be obtained by adopting a wider, flat segmental arch. For such extremes where the semi-circular is not satisfactory, it is possible to design a box section, and if necessary, a multiple box,

FIG. 1.—WORKING CONDITIONS FOR CONCRETE SEWERS.

though this latter should always be compared with a similar multiplication of normal arches before being adopted.

Where the sewer is deep, and in particular, if in rock, there is usually economy in making the heights of the section greater than the widths, and if in deep rock cut, it is possible to use plain concrete sides and a flat arch abutting on the rock.

Under average conditions the most economical section is undoubtedly one approaching nearly to the circle, that is, having width and height about equal, but on account of the difficulty of securing satisfactory construction

with a semi-circular invert, a segmental invert (usually a 45- to 60-deg. segment) has been common.

For loads due entirely to earth pressure and for sewers through fully developed territory where the loading can be definitely determined, arch sections of the semi-elliptical or similar types can undoubtedly be used to advantage, as the concrete can be worked in direct compression for the normal load and reinforcing put in to allow for unusual conditions. But where the loads cannot be predicted with reasonable accuracy or where extreme loads of opposite character must be provided for, there will be little difference between

FIG. 2.—UNUSUALLY GOOD WORKING CONDITIONS.

the semi-elliptical and semi-circular sections and there is undoubtedly a prejudice in favor of the latter.

The great advantage of the reinforced arch for use in sewer construction lies in the economy in material. Except in extreme instances where the box or slab construction is employed, an equally satisfactory sewer may be built of brick or brick and mass concrete, but in large sizes, 8 ft. or over, the reinforced concrete sewer requires only from 60 to 80 per cent as much masonry as the "gravity" type. Also where the amount of concrete per foot is sufficient to warrant an efficient plant, the unit cost of the concrete should be somewhat

less than that of good brick masonry. In addition to the saving in masonry, there is usually an accompanying difference in excavation and in deep work this may be a material consideration. Finally there may reasonably be a difference in required size of sewer due to the greater smoothness of good concrete work, which amounts to between 5 and 10 per cent reduction in mean diameter.

It has been the writer's experience that for sewers up to about 8 ft. in diameter, brick work is probably more economical than concrete and that for larger sizes the reverse is true, the saving in favor of concrete increasing with the size. This is, of course, in the light of local conditions and might be modified for other cities. The comparison is based on work in the open where sufficient ground is available for plant and material storage, and the statement must be again modified if the work is to be done in congested districts. As an example, it was recently considered economical to build a 14-ft. sewer of brick because of its location through a solidly built up residence district of the city.

There can be no question that reinforced concrete is the natural engineering solution for the problem of large sewers. If reasonably designed and carefully constructed, it gives the best and cheapest sewer. In the hands of a designer not thoroughly familiar with the conditions surrounding sewer construction and maintenance, or of a contractor not experienced in reinforced concrete work, it is likely to be a dangerous material and it is a much too common occurrence that work is handled under just these conditions. The fact that many of these sewers are built by contractors whose whole experience has been with massive masonry, has not tended to add to the safety of the finished work.

While some of the old sewer engineers have built reinforced sewers that would be reasonably safe and without reinforcement, there are many examples of exceptionally thin sections. These, the writer believes, are the result of assumptions that concrete can be placed in a sewer ditch with the same success as in a building and that the loading can be accurately forecasted. To eradicate this idea a brief description of average construction conditions may be warranted.

CONSTRUCTION.

There is no class of work in which so many difficulties surround the successful placing of reinforced concrete as in sewer work. Sewers are usually constructed along public streets or alleys at considerable depth below the ground, or if in shallow excavation, are likely to be along the general trend of some water course and may often cross and re-cross a running stream. For these reasons, the excavation is usually difficult and the sides of the trench hard to support. Probably the best work can be secured in the open country, where the sides of the trench can be sloped. In this case, there is no cross bracing to complicate the form work and outside forms can be used and removed. Where the sides of the trench must be vertical, it is necessary to put in horizontal planking and cross bracing as the excavation proceeds, and it is economy to use as little lumber as possible below the spring line of the sewer, as such

lumber is necessarily concreted in and lost. For the same reason, outside forms are not customarily used for the lower portion of the work.

When the excavation is complete, the invert is concreted. In rock or in dry ground, this can be done efficiently but if water or mud is present, a portion of the concrete is sure to be unsatisfactory. With very bad bottoms, it is often

FIG. 3.—CHUTING CONCRETE FOR SEWER.

necessary to place a raft of extra concrete and allow it to set before attempting to place reinforcement. If such conditions appear possible, good practice will provide for this work in both specifications and estimate and many reasonably provide for underdrains to relieve the new concrete of damage from water flowing from the trench ahead. Unless the specifications are to provide that the work is to be expensively delayed, it should be noted that there will be

quite an amount of walking over and across the new invert while the concrete is setting and exposed bars left for splicing are likely to be bent and jarred and their bond value in the invert concrete decreased. Also because of these stub bars, it is usually impracticable to protect that portion of the invert from dirt and rubbish. While it is generally the custom to leave the sides of the invert rough to furnish a bond with the arch, it is an open question whether the finishing of this concrete smooth is not the lesser evil, as it can then be thoroughly and efficiently cleaned before additional concrete is laid.

Before the arch forms are set, it is necessary to remove cross bracing up to the crown level, and it must be replaced with verticals bedded in the new invert and cross-braced above the crown. Even with the most careful work, this will produce some disturbances with the sides of the trench and may even allow a bulging of the side plank enough to protrude within the neat measurements of the sewer. To widen and rebrace the section from the surface down may be expensive and hazardous as well as disorganizing, and the engineer often faces the problem of modifying the section instead. The writer recalls instances where the contractors have even asked permission to fill the whole trench to the top of the sewer with concrete at their own expense, rather than to attempt the re-excavation, and the construction engineer must be able to decide whether the deficiency in thickness at the sides can be compensated in this manner.

Collapsible steel forms are usually favored for the arch, and if kept well cleaned and oiled produce the best interior surface, but well-made wooden centers carefully planned will result in more satisfactory work. The choice will usually depend on the contractor's organization and schedule, as greater progress with one outfit can be secured if the collapsible forms are used.

Under the conditions prevalent in this work, the setting and holding of the arch reinforcement in accurate position is especially difficult and the importance of accuracy is rarely appreciated by foreman and laborer. When properly set, the rods are difficult to hold during concreting, as it is often necessary for the men to stand on the reinforcing while spading the concrete. The cost of special chairs or holders for the reinforcement is usually well warranted.

The placing of the concrete is made especially difficult because of the double mat of reinforcing bars, which tend to break up the stream of concrete and to cause a separating out of the aggregate. The concrete is also likely to be lowered in quality by an almost unavoidable leakage of water. The concrete is also contaminated to some extent by earth and rubbish knocked from the surface into the forms. There occurs, also, even in the best regulated work, certain small slips of earth from between the side planking, and it is possible that portions of the clay or loam may be churned into the concrete before it can be cleaned out from between the tangle of reinforcement.

In view of the unavoidable construction contingencies inherent in this class of work, the writer would recommend to the designer the following prescription:

1.—Use the best grade of concrete and considerable excess of mortar.

2.—Do not work concrete at more than 450 lb., unless the construction conditions are to be exceptionally favorable.

3.—The concrete cover outside of the steel should be at least 2 in.

4.—Use a minimum thickness of concrete of about 9 in. unless the work is close to the surface, or is to be built under very favorable conditions, and increase this minimum and also the cover over the steel if the conditions are likely to be very unfavorable.

FIG. 4.—DIFFICULT CONDITIONS FOR REINFORCEMENT.

5.—Specify the setting of the reinforcement with especially designed holders. These might be made of cast iron and left in the concrete.

6.—If there is any possibility that the trench will be very wet or mucky, provide for a sub-base of concrete and provide means of keeping the trench work away from the work, if possible.

7.—To secure a concrete that will flow into place with the least assistance, a specification for a 2½ or a 3-minute mix should be seriously considered, as might also the use of hydrated lime. This would naturally result also in a denser and more waterproof concrete and might be a very considerable factor in prolonging the life of the reinforcement.

8.—Provide for a lining of vitrified brick for the invert, or at least provide an excess internal area to allow for such a lining at some later date. This is of more importance in maintenance than in construction, as under average conditions it is easier to obtain a reasonably smooth invert with the brick than to attempt to finish the concrete itself.

9.—Specify cold weather methods. Concrete can be placed satisfactorily and economically at even a zero temperature, if proper precautions are taken. It should be noted, however, that it is quite easy to overheat the finished concrete and to drive out a portion of the water.

In the St. Louis work, it has been customary to heat the water by turning exhaust steam into the water tank whenever the temperature goes below 40 deg. or whenever there is frost in the materials. In colder weather, steam coils are used in the sand storage piles and often in the piles of coarse aggregate. It is also customary in freezing weather to place salamanders inside the arches and to hang tarpaulins at each end of the unit constructed. The top of the sewer has generally been protected by a covering of tarpaulin or plank, on top of which manure is piled.

LOADING.

The loads to be considered are first, direct weight of the earth filling; second, horizontal or inclined pressures induced by the weight of this filling and the adjoining earth; third, pressures due to transmitted surface loads.

The relative values of these pressures will depend on the depth and size of the sewer and on the use to which the ground surface may be put.

Vertical Loads.—It is always safe and usually reasonable to design for vertical loads equal to the full weight of the superimposed earth. Recent investigations of small sewers and pipes have shown that, due to some arching action of the earth itself, the full dead weight is not always applied to the sewer. The allowable reduction, however, seems to be of little importance until the depth of the fill is at least equal to the width of the trench and would only amount to about 25 per cent when this depth is twice the width. The work of Marston and Anderson indicates that for depths of 10 to 15 times the width, only 30 to 40 per cent of the load is carried by the sewer. For a sewer more than 8 ft. in width, the depth of cover will rarely exceed twice the trench width, so that the reductions are hardly worth taking into account. There must also be reasonable doubt whether the gradual settlement does not finally increase the weight on the sewers considerably above the values given.

Horizontal Pressure.—There is so much doubt as to the correct values of horizontal pressures, even for a given soil condition, and the pressures will vary so greatly in the different soils that the designer can only attempt to make a safe guess at the correct amounts to be used.

According to Rankin's theory, the intensity of horizontal pressure cannot be less than one-third of the intensity of vertical pressure for a particular depth and in ordinary clay it is customary to consider it as one-half of the vertical. For saturated ground, the earth will approach the condition of a fluid and the horizontal and vertical pressures would be equal.

FIG. 5.—OBSTACLES TO CONCRETING FROM BRACING.

Surface Loads.—Where sewers are constructed in city streets, the heaviest surface load would be the weight of a road roller, and this might be taken as 15 tons on an area of 5 sq. ft. at the surface, distributed downward along an angle of 30 deg. with a vertical. At a depth of 10 ft. this would approximately

be equal to 200 lb. per sq. ft. on an area of 11 x 15 ft., or roughly equivalent to an additional 2 ft. of fill. If there are railroads crossing the line of the sewer, or if it seems at all possible that such roads may be built, the sewer should be designed for locomotive loading in the same way. A fair value for this loading would be 80 tons on an area of 10 x 20 ft. at the surface. Distributed as above, this would be equivalent to about 300 lb. per sq. ft. over an area of 20 x 30 ft. at a 10-ft. depth and would give the same pressure as 3 ft. of additional fill.

For very light covers, these values would, of course, be increased, and it might even be reasonable to provide for impact, but for depths of cover for

FIG. 6.—FOUNDATION WORK FOR A SEWER.

6 ft. or more, it is usually satisfactory to treat such loads as additional weight of earth and allow them to increase both the vertical and the horizontal pressures. Allowance for foundations and for piles of material may be handled in the same manner.

Combination of Loading.—For final conditions, that is, after the backfill has reached a state of settled equilibrium, the sewer will be subject to a direct combination of horizontal and vertical pressures. It should be noted that the greatest bending moments in the arch will be due to vertical loads alone. Horizontal pressures usually induce moments of the opposite kind. The combination of vertical and horizontal pressures, therefore, while increasing the direct normal compression in the arch, will give smaller bending moments

than those from the vertical loads. While the stress in the arch may finally reach the values derived from a proper combination of the two classes of forces, yet it is quite common for the sewer to be subject only to pressure of one kind during the construction period. Examples of this are as follows:

(*a*) A trench is excavated through hard clay which requires little bracing and will stand vertically for some time. The trench is backfilled with the same material. Then the full weight of the backfill may act vertically on the arch for some time before the sides of the trench finally slip and add also horizontal pressure.

(*b*) In the example above, the sides of the trench may slip in against the sewer before the backfill is placed, producing heavy horizontal pressure and bending moments of reverse character.

(*c*) A trench through soft ground is held by sheet piling. When this piling is pulled there may be an appreciable time before the earth at the sides closes in and fills the void left by the piling. During this time the vertical loads only will act.

(*d*) In the above example, if the sheet piling is drawn before the back-filling is started, the earth at the sides may move in and produce horizontal pressure with very little vertical load.

Loads of these kinds will only occur while the arch is new, possibly before the concrete has attained more than half of its normal strength. If the design contains a factor of safety of four for combination of pressures, and the concrete is only 10 or 15 days old, the arch would be about on the point of failure for vertical loads.

It would seem, therefore, that the design should provide for vertical loads alone, or at least in combination with a very small horizontal pressure on the arch only (not against vertical side walls). This loading will be critical and from it the dimensions of the concrete and one set of reinforcements will be determined. The arch so determined should then be designed for horizontal pressure in combination with as little vertical loads as may seem possible. From this the reverse reinforcement may be calculated. Finally it is of interest to compute the stresses under normal combination of the two.

DESIGN.

The simplest case of arch design occurs when the sewer is built in a rock cut. In this instance, that portion above the rock may be taken as an arch with fixed ends, provided that the reinforcement extends well below the rock level. Where the sewer rests on rock or other incompressible material, the arch may still be treated as fixed, if sufficient mass is given to the invert to resist the overturning moment in the side walls.

If the sewer is constructed in soft or compressible soil, the whole section, including the invert, should be treated as an elastic bent beam and the loading must include an upward pressure on the invert equal to the total vertical load.

A number of methods have been published for the analysis of the elastic arch. Of these the simplest is that presented in Green's "Trusses and Arches." Professor Green worked out bending moments in the parabolic arch for unit

loads. He also presented constants for the semi-circular arch. Green's constants for the semi-circular arch have been extended by Mr. A. E. Lindau (*Trans. Am. Soc. C. E.*, Vol. 51), and put into the same tabular form as was originally given for the parabolic arch. Green's analysis is based on a constant ring thickness. It is not correct for the usual case in which the arch increases in thickness from crown to springing line and some idea of the error involved is given in Lindau's paper. Although inaccurate, this method is very convenient, in that by its use we are able to calculate moments directly from the loading without the previous assumption of an arch thickness. For the smaller sewers and not for all purposes and where the variation in ring thickness is not great, it is sufficiently accurate.

In the writer's practice, this method has been developed into a set of formulas applicable to the semi-circular arch. These formulas give the moment at each 10 deg. point in terms of the mean radius of the arch and of the depth of fill over the crown. In the more important work, these formulas are used in order to determine approximate dimensions for an arch which is later to be analyzed by one of the more accurate methods. As the accurate methods must be applicable to all shapes of arches and variations in thickness, it is impossible to reduce them to any very simple form.

A full theoretical description of the subject is given in Howe's "Symmetrical Arches" and a simplified description and method of application is presented in Turneaure and Maurer's "Principles of Reinforced Concrete." The best practical summary of all methods and several excellent examples are to be found in Metcalf and Eddy's "American Sewerage Practice," Vol. I.

In detailing the arch from the calculated bending moments, it will usually be found advisable to use two full sets of reinforcement, that is, on the inner and outer face. If it were known positively that reverse moments could never occur, for example, if it were impossible in the case of a semi-circular arch that the horizontal force could predominate, it would be reasonable to omit a portion of one set of reinforcement or possibly to cross one set over from the inner to the outer face, but this generally cannot be insured and the full reinforcement should be put in even if only as an added factor of safety and for the sake of standardization. Another reason why the arch cannot be designed too closely is that any particular section, if multiplicity of sections is to be avoided, must be designed for variations of loading over a considerable range.

In the St. Louis work, where a considerable length of one size sewer and fairly constant soil conditions occur, it has been the practice to design a section for each 5 ft. in depth of loading and to detail these sections for soft ground foundation, for hard bottom and for deep rock cut. A designer cannot follow too closely the calculated thickness of the arch, as some consideration must be given to the shape of the outside as well as the inside of the sewer. For example, if the sewer is to be built in a trench with vertical sides, it would be found much simpler to make the outside of the sewer vertical to some point above the spring point of the arch rather than to carry a small batter all the way down to the bottom of the sewer. This is because of the fact that it would cost less to fill in the small wedge-shaped space with concrete than to attempt to place and remove outside forms in the limited space available.

There seems to be no uniformity in practice as to the longitudinal reinforcement. A certain amount of steel is usually required in this direction to properly tie in the transverse bars and $\frac{1}{2}$ or $\frac{3}{4}$-in. bars are often used on about 2-ft. centers in both faces. If the sewer is to be constructed in hot weather and particularly in shallow cut, it might be advisable to increase the amount of longitudinal steel in order to distribute shrinkage cracks, but under other conditions this seems hardly necessary as the range of temperatures in the

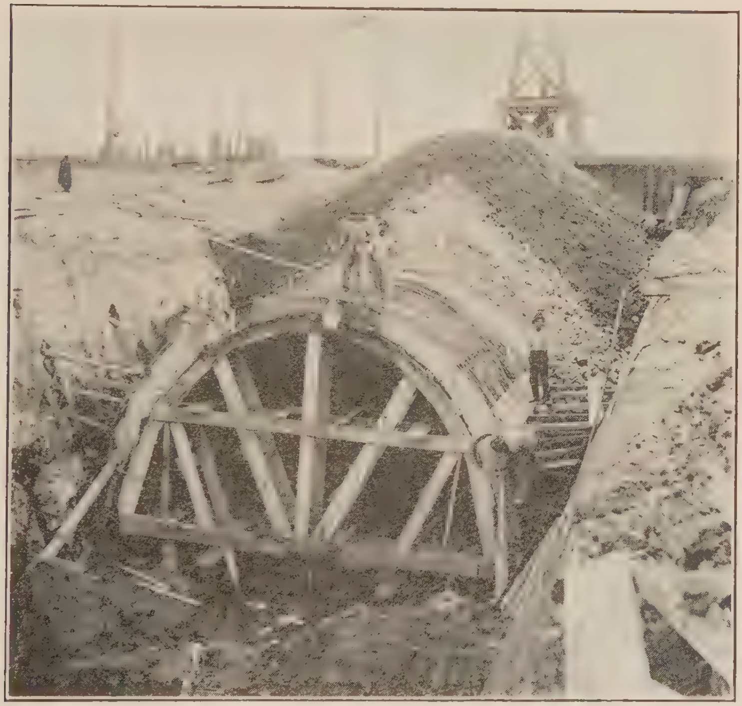

FIG. 7.—A WET TRENCH.

completed sewer is very small, probably varying from about 40° F. in winter to 70° F. in summer, unless steam or hot wastes are permitted to enter.

It is assumed that any elaboration along the line of the mechanics of reinforced concrete would be superfluous before a Society of this character, and the writer has endeavored to outline from the point of view of the sewer engineer, the conditions which should be considered in determining the loading of reinforced concrete sewers and certain other points of importance on account of the conditions surrounding the finished structure.

CONSTRUCTION METHODS ON THE TUNHANNOCK AND MARTIN'S CREEK VIADUCTS, LACKAWANNA RAILROAD.

By C. W. Simpson.*

Two large concrete structures, known as the Tunkhannock and Martin's Creek Viaducts, have recently been completed by the Delaware, Lackawanna and Western Railroad Company, in connection with a grade improvement between Clark's Summit and Hallstead, Pa. As the design and structural features of these viaducts have been extensively described in the technical press, this paper will be limited to a brief outline of the structures and to pointing out a few of the more essential features of the planning and carrying out of the work of construction.

Martin's Creek Viaduct.—The Martin's Creek Viaduct is a three-track bridge having a maximum height above stream-bed of 150 ft., and a length of 1600 ft. It has two 50-ft. and two 100-ft. semi-circular arches, and seven 150 x 59-ft. three-centered arches. The larger arches are 6 ft. thick at the crown and are divided into two ribs, 17 ft. 6 in. wide, spaced 29 ft. 6 in. center to center, which carry transverse spandrel walls supporting an arched floor system. This structure required about 26,000 cu. yd. of foundation excavation, 77,000 cu. yd. of concrete and 1,600,000 lb. of reinforcing steel. The completed bridge is shown in Fig. 1.

Tunkhannock Viaduct.—The Tunkhannock Viaduct is a two-track bridge composed of ten 180-ft. and two 100-ft. semi-circular arches springing from solid piers and supporting transverse spandrel walls, upon which rests a floor system composed of 13-ft. 6-in. semi-circular spandrel arches. The deeper piers have a section 40 x 46 ft. below the ground surface and all piers are 36 ft. 6 in. by 43 ft. 6 in. to a point 17 ft. below the springing line of the main arches. At this elevation a 4-ft. 3-in. offset provides a seat for the temporary steel centering. The deepest pier extends 103 ft. below the original ground surface, and at this pier it is 309 ft. from the bottom of the foundation to the highest point of the masonry. Each main arch is composed of two ribs, 8 ft. thick at the crown and 14 ft. wide. About 163,000 cu. yd. of concrete, 2,500,000 lb. of reinforcing bars, and 48,000 cu. yd. of foundation excavation were required. All piers were carried to solid rock, the depth of which had been determined by borings. The completed bridge is shown in Fig. 2.

The natural conditions at the site of each of these structures and the method of carrying out the construction work (with one exception) were so nearly identical that a description of the Tunkhannock Viaduct will apply equally as well to the Martin's Creek. The exception noted is that the

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Martin's Creek bridge was constructed entirely with derricks, while at the Tunkhannock Bridge both derricks and a cableway were used. The reason for the use of a cableway at the latter structure was its height and the length of the spans.

GENERAL PLAN OF PROCEDURE.

The contract for this work was let in June, 1913, and the time set for completion was July 1, 1916.

In planning the work no detailed schedule was laid out, but the following general plan of procedure was determined upon.

Of the thirteen foundations, six were more than 40 ft. deep, and steel sheeting was adopted for these six excavations. At three of these and at three of the other foundations, the configuration of the ground was such that a portion of the excavation could be accomplished with a steam shovel. Of the other three deep foundations, two were partly in the bed of the stream and were considered to be the most difficult. It was decided to provide sufficient sheeting for these two piers; to start them at the earliest possible date; and to re-use this sheeting at the other four deep excavations. In the meantime the steam shovel excavation was to be carried out and the other shallow piers excavated.

In planning the concrete schedule it was necessary to consider that the mixing plants would be on low ground while five of the shallow excavations were in much higher ground and could be reached economically only by the use of the cableway. For this reason no large amount of concrete could be placed until either the cableway or some of the deep foundations were completed. It was estimated that of the excavations accessible without the cableway, four (two shallow and two deep) could be completed on or before January 1, 1913, and the concrete schedule was made out from that date, as the cableway could not be ready for use before February.

These conditions made it necessary to contemplate the placing of 163,000 cu. yd. of concrete in 30 months. It was, of course, recognized that concreting could be carried on much more rapidly and continuously while working on the foundations and on the main pier shafts than would be the case when working on the arch rings and spandrel system. About half the concrete, or 80,000 cu. yd., is below the springing line of the main arches, and it was decided that this quantity must be placed during the first ten months, or at the rate of 8,000 cu. yd. per month. This made the rate for the remaining twenty months 4,200 cu. yd. per month.

It must not be understood from this that it was the intention to complete all the work below the springing line before proceeding with the superstructure. On the contrary, it was desired to combine work on the superstructure with the part below the springing line as much as possible; but it was realized that the pier work must predominate during the first third of the allowed time. Also, it was not expected to maintain a uniform monthly rate. The schedule was simply a general guide as to the sufficiency of the plant and forces engaged.

FIG. 1.—MARTINS CREEK VIADUCT, LACKAWANNA RAILROAD.

The organization of forces was as follows:

A general manager was in complete control. To him reported a superintendent and an office manager. A timekeeper and a material clerk reported to the office manager regarding office matters and to the superintendent regarding field matters. The superintendent's assistants were as follows: master mechanic, general carpenter foreman, foreman rigger, foremen of concrete gangs, foremen of excavation gangs, mixer foremen, foreman of material gang and foreman of steel gang. The master mechanic was in charge of all machinists, enginemen, hoist runners, cableway runners, steam shovel men, pumpmen, drill runners, blacksmiths, signal men and electricians. The carpenter foreman had under his direction sub-foremen, mill-men, carpenters and helpers. The rigger foreman was responsible for the condition of the cableway and derricks. The duties of the other foremen are evident from their designation. Laborers and sometimes foremen were shifted from one class of work to another as occasion demanded, but all men were kept at the same work so far as possible.

GENERAL LAYOUT.

Before the completion of the improvement the main tracks of the Delaware, Lackawanna and Western Railroad were approximately parallel with and about 450 ft. distant from the viaduct. This is shown in Fig. 3 (Plate I), which shows the general plant layout. (It will be noted that this plant is to scale between piers 3 and 8 only, the abutments and piers 1, 2, 9, 10 and 11 being omitted, as no plant was located between the piers shown and the end cableway towers.) The railroad company's tracks were on an embankment, so that they were about 60 ft. higher than the creek bed and about 35 ft. higher than the ground level at the mixing plants. The loop formed by the narrow gage track encloses a knoll about 30 ft. higher than the general level of the valley. On this knoll are located the machine and blacksmith shops, superintendent's office and water tower. The general office and hospital are located on ground at about the same elevation as the knoll above mentioned. The general material derrick is located on a narrow ridge which connects the railroad embankment with the knoll at the blacksmith shop. A similar ridge extends from pier 6 to the high ground at the hospital. Cuts were made through these ridges to permit the laying of the 3-ft. gage tracks.

The first operation was the widening of the railroad company's embankments and the laying of the track marked "Track for Empties" and "Room for 16 Loads." This track was completed on June 12; a track pile-driver immediately started to construct the material trestles; the general material derrick was erected; work was commenced on the shops, storehouses and office building; and a narrow gage track was laid from the material derrick to pier 4. During July a steam shovel started foundation excavation, and sheet piling was being driven at pier 4. During August the erection of the cableway was started; a trestle carrying a narrow gage track was built across the creek to give access to pier 3, and a large amount of plant and supplies were delivered on the ground.

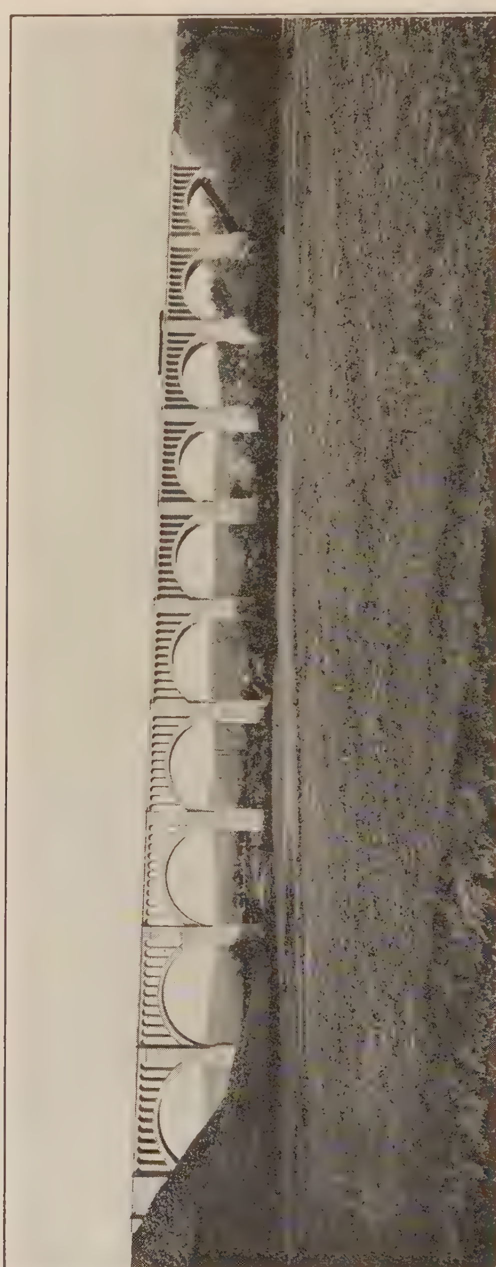


FIG. 2.—TUNKHANNOCK VIADUCT, LACKAWANNA RAILROAD.

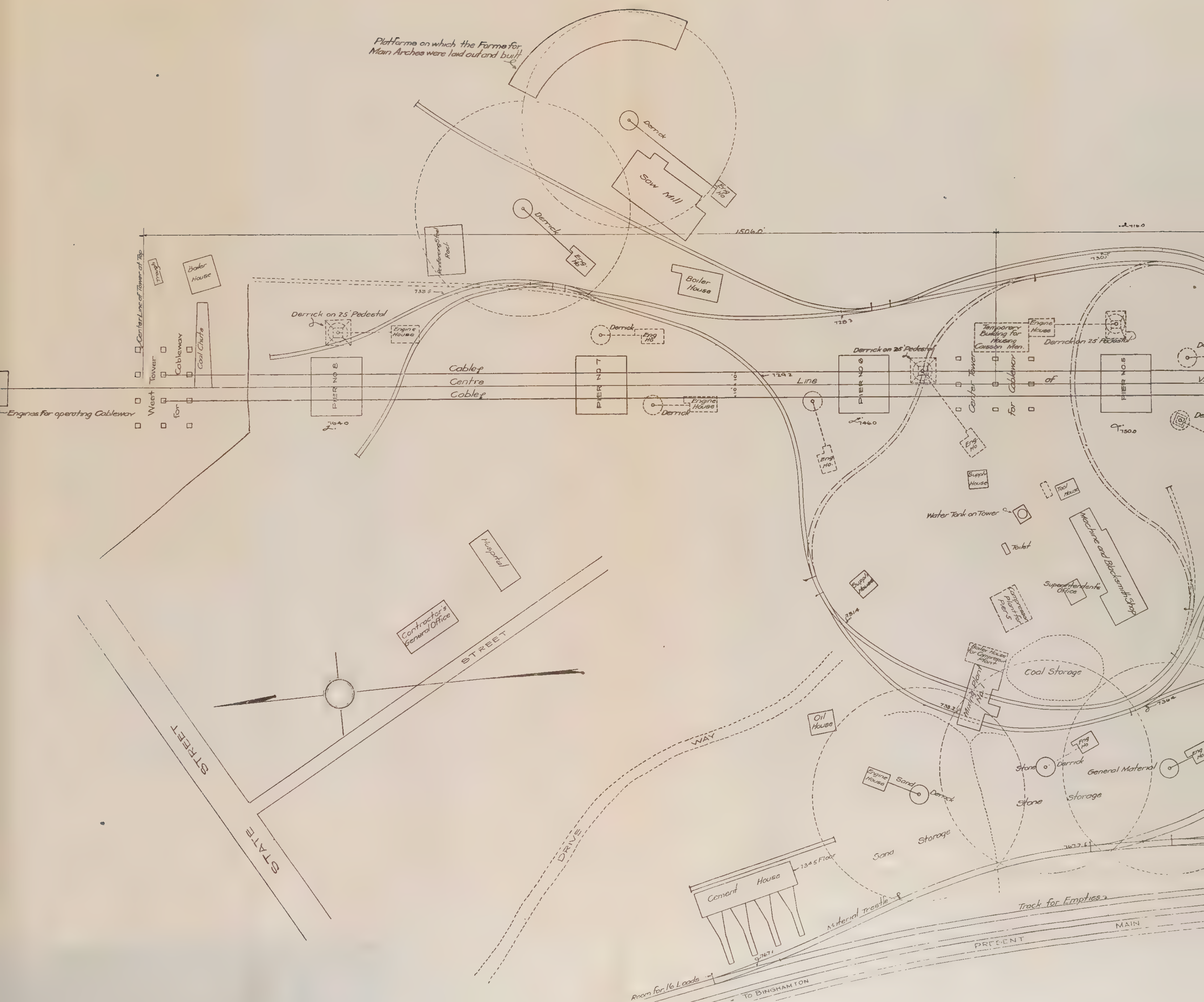
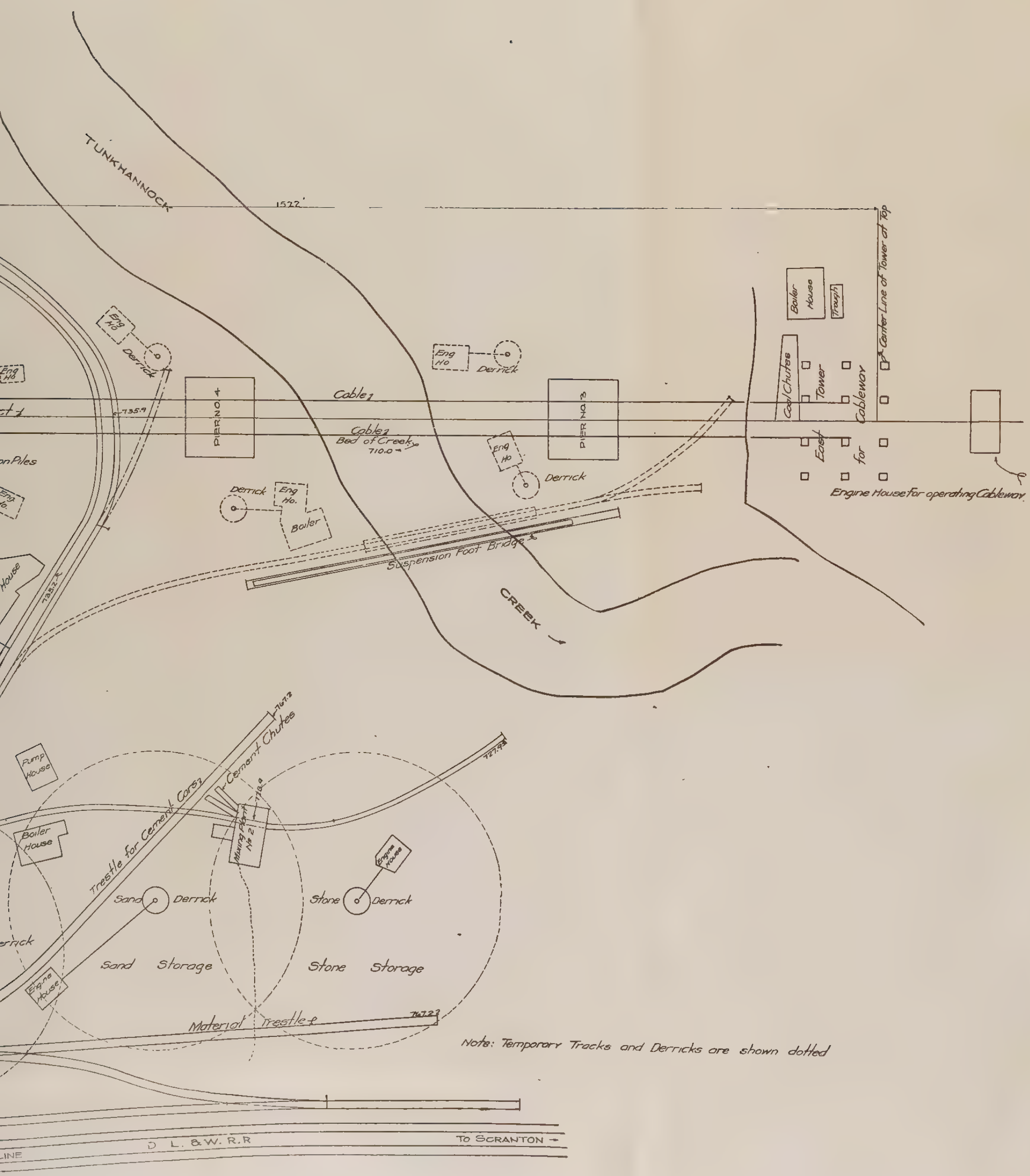


FIG. 3.—LAYOUT OF CONSTRUCTION PLANT.

Plate I.
Proc. Amer. Conc. Inst.
Vol. XII, 1916
Simpson on Construction of
Tunkhannock Viaduct.



FOUNDATION EXCAVATION.

A steam shovel having a 1-cu. yd. dipper excavated piers 5, 6 and 8 down to ground-water level and piers 2, 9 and 10 down to rock. The material was allowed to take its natural slope, and the excavations were made sufficiently large to take care of the raveling of the banks prior to the completion of the piers. The shovel loaded the material into 4-cu. yd. cars, and it was utilized in grading for the narrow gage tracks and in bringing all low ground between the creek and the railroad up to a level with the mixing plants. All the ground around the sawmill, laying-out platform, steel rack, boiler house, and mixer 2 was "made" in this way. This "leveling up," amounting to as much as 12 ft. in some places, was done at practically no extra cost and proved a very great benefit in handling the work. The shovel completed its portion of the excavation in December, 1912, having excavated about 50,000 cu. yd., 15,000 cu. yd. of which was pay material.

In excavating below water level, interlocking steel sheet piling in lengths of 30 ft. were used. The sheeting was driven with a 3-ton steam hammer and was braced with 12 x 12-in. timber. Sheeting was required for depths of from 50 to 75 ft., necessitating the use of two and three lengths of sheeting.

The method of procedure with the excavation was to erect one set of sheeting on lines about 3 ft. outside of the required foundation area, and to drive the sheeting as far as convenient. The excavation was then carried down to the bottom of the sheeting by means of $1\frac{1}{2}$-cu. yd. clam-shell buckets, operated in the bays between the timber braces, which were so spaced as to admit the operation of the bucket. The bracing was put in on about 5-ft. centers, vertically, as the excavation proceeded.

After the excavation had been carried down to the temporary bottom of the sheeting, the latter was driven farther, and the operation was repeated until the top of the sheeting was down to the ground surface. Another set of sheeting was then erected about 6 ft. 6 in. outside the first set, and was driven as deep as was thought advisable. The material between the two sets of sheeting was excavated with the clam-shell bucket; the first sheeting was then driven farther; and the excavation was resumed. As the inside set of sheeting was driven down, the upper set of timber bracing was removed and placed at the bottom. The process was continued until bedrock was reached. Almost all of the sheeting was recovered, some of it being used four times.

Two of the piers gave serious trouble on account of quicksand. The difficulty was overcome in one of these by dividing the last 12 ft. of the excavation (the total depth was 62 ft.) into three pockets and taking each pocket down separately. A somewhat similar plan was tried in the other excavation but was not successful, and it was necessary to resort to compressed air. A caisson was built inside the sheeting with its cutting edge 45 ft. below the top of the excavation and 32 ft. above bedrock. The two lower sets of bracing timbers were built into the caisson and were carried down with it. Concrete was placed on top of the caisson as it sank in the usual manner. Had it not been for the difficulties that had to be overcome in excavating for this pier, the work would have been completed well within the contract time. This

excavation was not completed until February, 1915, and yet the entire bridge was completed in September, less than three months later than the contract time.

CONCRETE PLANT.

Sand and stone were delivered on the material trestles in bottom-dump hopper cars and dumped into the storage piles. Each mixing plant was served by two derricks which operated clam-shell buckets of 40-cu. ft. capacity. These derricks conveyed the sand and stone from the storage piles to small bins over the mixers. From these bins the material passed by gravity into a measuring hopper where the cement was added. From the measuring hopper the material passed by gravity into a 2-cu. yd. mixer which dumped directly into double-line bottom-dump buckets on flat cars. Twelve-ton locomotives hauled trains of three flat cars, one car being empty to receive the empty bucket from derrick or cableway. At mixer 1 the cement was conveyed from the original cars on the material trestle to the hopper floor of the mixer by the sand derrick. At mixer 2 the cement was unloaded from the cars into chutes, which conveyed it to the hopper floor. All cement was shipped in cloth. A cement house (see Fig. 3) of 3000 bbl. capacity was built and filled, but no cement was used from this source except when shipments were delayed and no cement was available in cars.

The mixing requirements made it necessary to turn each batch from 2 to 2½ minutes and limited the mixer output to an average of 17 batches per hour. Each mixer gang consisted of one foreman, two derrick operators, one mixer runner, one fireman and eleven laborers. Two trains, each requiring one engineman and one laborer, served each mixer. One foreman with from eight to sixteen laborers spread the concrete in the forms. The above organization, with the addition of cableway runners and signal men, averaged about 30 cu. yd. of concrete per hour.

The following table gives the cubic yards of concrete placed during each month. It is interesting to compare this with the original schedule.

	1913	1914	1915
January	8410	3400	2000
February	5740	2260	5130
March	4520	1460	5620
April	6630	5090	4010
May	7190	3270	3200
June	13890	4010	3400
July	12150	4770	4000
August	8810	4450	1800
September	3820	4910	
November	3270	4310	
December	2160	4000	

About 8900 car loads of sand, stone and cement were handled on the material trestles and 1200 car loads of other material were handled by the general material derrick.

FIG. 4.—CABLEWAY TOWERS, ETC.

DERRICK AND CABLEWAY.

Derricks were operated by three-drum, $8\frac{1}{4}$ x 12-in. hoisting engines with swinging gear attachments. Masts were about 90 ft. and booms 85 ft. long.

FIG. 5.—TYPICAL CENTERING.

In a few cases derricks were erected on timber pedestals or pile clusters about 25 ft. high, but in most cases they were set at the ground level, and all work above their reach was carried on by means of the cableway.

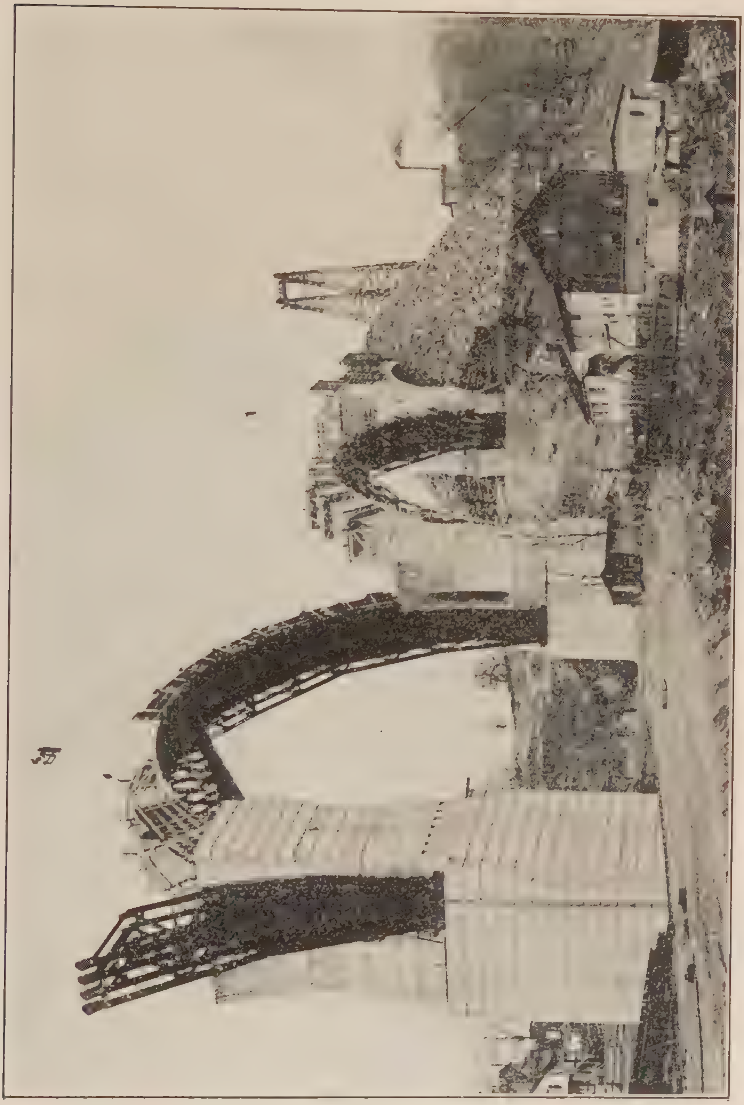


FIG. 6.—ERECTION OF CENTERING.

The cableway consisted of two lines of $2\frac{1}{4}$-in. main cables, spaced 20 ft. apart and supported by end towers about 160 ft. high and an intermediate tower 300 ft. high. The end towers were 3028 ft. apart and the intermediate tower divided this distance into two spans of about equal length. Four engines and carriages operated as four independent units. The towers were of timber and the intermediate tower was so located and designed that the arch rings and all the floor system except a narrow slot could be constructed through it and thus utilize the cableway to the fullest extent. The type of tower construction is shown in Figs. 4 and 5.

The last centers to be erected were those passing through the center tower. The adjacent piers were carried up to the top of the floor before

FIG. 7.—CENTERING WITH TIMBER TOWER.

these centers were erected and two stiff-leg derricks were located on each pier. After these last centers were erected, all material was raised above them by the derricks and then transferred to the cableway for transportation to its destination.

Arch Centers.

Five sets of three-hinged steel arch centers were used in constructing the 180-ft. spans. Each set was composed of four ribs weighing 47 tons per rib, spaced 3 ft. 10 in. center to center, and provided support for one of the two concrete ribs composing each span. A 4-ft. 3-in. ledge or offset, 17 ft. 6 in. below the springing line of main arches, was provided to support the centers. On this offset rested an I-beam grillage extending the full width of the pier

and carrying 6-in. rollers on which the pedestals of the centers rested. For erection purposes each steel rib was built in four sections.

Before the centers were erected the adjacent piers were constructed to an elevation about 37 ft. above the springing line, forming what is termed the "umbrella." The procedure of erection was as follows: After the I-beam grillage, rollers and pedestals were in place, the lower quarters of the ribs were raised to position, the bottom pins driven and the top of the sections anchored to the "umbrella" tops by bolts passing through the concrete. One of the upper quarters was next raised to position and bolted to the quarter already in place. The other upper quarter was then put in position, bolted to its lower section and the crown pin driven. The ribs were so designed that the half rib would support itself as a cantilever. Fig. 6 shows two sets of centers in place and the lower quarters of another set erected. Fig. 5 shows the upper quarters being erected through the center tower.

To provide for striking the centers or adjusting the crown elevation, the first panel on each side of the crown pin was constructed with pin connections, and the web member at right angles to the chords consisted of two parts connected by a right-and-left thread screw operated by a lever and ratchet. By lengthening or shortening this member the distance between the crown and end pins was decreased or increased, thus lowering or raising the crown.

After serving their purpose under one rib of a span, the centers were slacked off and jacked over to their position under the twin rib. When the span was completed, the centers were rolled back under the opening between the concrete ribs and transported to their next point of service. In constructing the last two spans, centers were erected under both ribs simultaneously in order to hasten the completion. Timber centers resting on a timber tower were used for the 100-ft. spans. The type of construction of these centers is shown in Fig. 7, which also shows the placing of lagging on a set of steel centers.

FORMS.

With very few exceptions the forms were built on the ground in sections and hoisted to position by cableway or derrick. These sections were re-used a great many times, some of the first ones built being still in service at the end of the work. To illustrate the construction and utility of this type of form, a description of the forms used for the main pier shafts will be given in some detail. These sections were of two sizes, one being 18 ft. 3 in. and the other 15 ft. 8 in. long, and both were 17 ft. 9 in. high. The larger section weighed about 7000 lb. Four of the larger and six of the smaller sections were required to surround one pier and were termed a set. Four sets sufficed to construct all the piers up to the centering ledge without in any way retarding the progress of the work and, with slight alterations, these same sections were used up to the tops of the piers and also as spandrel wall forms. These forms are shown in Figs. 4 and 5.

Each section is made by nailing two layers of 1 x 8-in. tongue-and-groove boards to 8 x 10-in. horizontal studs spaced 2 ft. 5 in. on centers, one layer being at right angles to the other and both layers at 45 deg. with the studs.

The studs were bolted to 10 x 10-in. verticals which extended about 2 ft. above the planking. All forms were faced with No. 26 galvanized sheet iron and, in some cases, paper was laid between the layers of boards as an aid in protecting the concrete during freezing weather. The forms were kept in position by rods running down at an angle of 45 deg. from the vertical posts to anchors in the concrete and by 12 x 12-in. timbers resting on the top edge of the planking and wedged against the verticals. The rods were provided with a threaded sleeve joint so that the projecting end could be removed from the concrete.

Six carpenters with two cables or one derrick would remove and re-erect one set of forms in somewhat less than two days. On two occasions when the plant was available and it was desired to move the forms with as much speed as possible, 16 carpenters, with the aid of two derricks and two cables, removed and re-erected a set of these forms in seven hours.

The forms for the arch rings were very similar in construction to the pier forms, and two sets, that is, forms sufficient to cover two complete ribs, sufficed for the entire work.

The main arches were constructed with large blocks or vouissoirs separated by small keys. The block forms were made entirely separate from the key forms. Fig. 6 shows the key forms in place on one span and the block forms being erected on the adjacent span.

These structures were designed and built under the direction of Mr. G. J. Ray, Chief Engineer, and Mr. F. L. Wheaton, Engineer of Construction. Mr. A. B. Cohen was in charge of the design and the writer was Resident Engineer in charge of the construction of the Tunkhannock Viaduct.

The contractors were Flickwir & Bush, Inc., for whom Mr. F. M. Talbot was General Manager and Mr. W. C. Ritner, Superintendent.

THE FALL RIVER CONCRETE CONDUITS.

By Frederic H. Fay.*

This paper is the very brief statement of a complicated municipal problem in which are involved many questions of sanitation, of stream control, of mill engineering, and of city planning, and to which a concrete conduit of peculiar design furnishes the key.

Fall River, Mass., the largest cotton manufacturing center in the United States, is a city of about 125,000 population, located on the seacoast on a tributary of Narragansett Bay, some 50 miles south of Boston. Its location is most favorable for manufacturing; in addition to ample rail facilities, there is a good tidal harbor on the west of the city, while to the east, and within two miles of City Hall, lie the Watuppa Ponds, which are at an elevation of 130 ft. above sea level and furnish water for both domestic and manufacturing purposes.

The North and South Watuppa Ponds, originally one large lake, 8 miles long and $5\frac{1}{4}$ square miles in area, constitute the largest body of fresh water in Massachusetts. The lake has been divided into two parts by the building of a causeway or dam at the Narrows. These ponds drain a watershed $27\frac{1}{2}$ sq. m. in area. The total storage capacity of the two Watuppa Ponds is 15,000,000,000 gal., while about 4,000,000,000 gal. in addition are stored in three tributary ponds lying to the south. Only a fraction of this total storage is now available, however, since it is not possible to draw these ponds down to any considerable depth.

The North Pond is chiefly used for the domestic water supply of the city. The water of the South Pond and of the three tributary ponds, with a certain amount of North Pond water which is drawn into the South Pond, is used for mill purposes. About one-quarter of the total yield of the watershed is used for the domestic supply and three-quarters for industrial purposes.

Quequechan River.

The outlet of the Watuppa Ponds is the Quequechan River (the Indian name, "Falling Water"), which flows from the South Pond through the heart of the city. For the first two miles, from the pond to a point near City Hall, its course is almost level, thence in the last half mile it drops 130 ft. through a succession of falls and empties into the harbor known as Mount Hope Bay. The name of the city was derived from this stream, and from earliest times the water power of the stream has been utilized by mills along the lower half mile of its course.

For the first two miles of its course the Quequechan was originally narrow, winding through meadows. Ninety years ago, the mills at the various falls, desiring to obtain greater storage of water for power purposes, combined

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under a charter from the Legislature of Massachusetts and built a dam at the uppermost fall, which raised the level of the water in the stream and in the Watuppa Ponds to a height about 5 ft. above the natural high-water mark. As a result, some 200 acres or more of the flat meadow land bordering the upper two miles of the stream were overflowed to a shallow depth.

With the introduction of the steam engine, mills were built around this upper river basin to use the water for boiler feed and condenser purposes. The city has grown around these mills until today the 200 acres of shallow flowed flats are in the very heart of the community.

Of a total of 4,100,000 spindles in all the cotton mills of Fall River, nearly two-thirds, or 2,638,000 spindles, are found in the mills along the $2\frac{1}{2}$-mile stretch of the Quequechan River. Of these latter spindles, about 2,000,000 are in the 27 mills surrounding the upper basin. Today water power is a negligible factor, the river water being used almost wholly for condenser and other industrial purposes.

The sewers of the city have not kept pace with the growth of the population of this district, and for years the sewage from most of these mills and from a considerable district bordering the basin has been discharged into the Quequechan. In addition to sewage, the stream is polluted by the discharge of bleachery, dye and other mill wastes.

During wet months, when water is abundant and the ponds and Quequechan basin are filled nearly to high-water level, conditions are not particularly objectionable. During the dry months, which usually occur in the summer and fall, however, the water of the ponds and basin is usually drawn down 3 or 4 ft., exposing flats covered with an obnoxious sludge consisting of sewage matter, silt and mill wastes. Also, at such times the mills are seriously hampered by lack of sufficient water for condenser purposes. Those bordering the upper basin require some 30,000,000 gallons per day, while the flow of the stream is often reduced to 5,000,000 gallons; the result is that the water is drawn from the stream and returned to it again after being passed through the condensers of mill after mill, until frequently the entire flow of the river becomes heated to 100° or even 120° F. and has little value for the cooling of condensers.

So far there has been utter lack of co-operation among the mills and no policy of conservation in the use of this water. The result is that there are low water periods nearly every year when the mills suffer for lack of water and conditions in this thickly-settled community become highly objectionable from the standpoint of health.

The Remedy.—For many years the need of improving conditions along the Quequechan River has been recognized by the city of Fall River and several tentative plans have been suggested. Recently, however, the whole situation has been most thoroughly studied by a local municipal commission appointed under authority of a state legislative act and a remedy has been proposed, the acceptance of which is now under consideration by the city authorities.

The proposed scheme of improvement will completely abate the present nuisance and, by the development of 100 per cent increase of storage capacity

in the South Pond, will so regulate and utilize the flow that the mills will have an ample supply of water at all times.

In general, the proposed improvement provides for the following:

1. The construction of a large reinforced concrete main conduit, nearly two miles in length, which, with its branches, will provide:

(*a*) An adequate supply of clean, cold water to all mills along the stream;

(*b*) For the return of the hot water discharged from the condensers of the mills above the Watuppa Dam to the South Pond, where, after purification and cooling, it will become available for use again;

(*c*) For the discharge into an outfall sewer of the surface water which now flows into the Quequechan River; and

(*d*) An adequate channel, by which, in times of flood, the waters of the Watuppa Ponds can be drawn off and discharged into the harbor before they rise to a level that would cause damage.

2. The construction of a system of reinforced-concrete drains to collect and bring to the main conduit the surface drainage of the Quequechan valley watershed.

3. The construction of a system of reinforced concrete sewers to care for the sewage of the present unsewered district along the river.

4. The construction of a large outfall sewer, one mile in length, by which both the sewage and storm drainage thus collected will be taken to the water-front of the harbor.

5. The filling of the entire area of the Quequechan River flats, some 200 acres in extent, thereby not only abolishing the nuisance which is always present at low water when the sewage-covered flats are exposed, but also making available for industrial or other uses a large tract of land in the heart of the city, the value of which will offset in part the cost of the proposed improvement.

THE THREE-LEVEL REINFORCED CONCRETE CONDUIT.

Investigation of a number of possible schemes showed that a satisfactory solution of the problem could be reached only by confining the river within an artificial channel and filling the entire area of the shallow-flowed flats. A concrete conduit was finally selected as the proper type of river channel. Furthermore, in order to develop greater storage capacity in the South Watuppa Pond, it was found essential to place this new river channel, the cold-water conduit as it is called, at a level considerably below the present river bed.

As the mills along this two-mile stretch of conduit require water for condenser purposes to an amount considerably greater than the normal average flow of the stream, a separate channel was planned by which the hot water discharged from these mill condensers could be taken back to the South Watuppa Pound for use again. It was found that if this hot water conduit is at sufficiently high elevation, the condenser water will return to the pond by gravity. The hot water conduit was placed directly over the cold water conduit, thereby avoiding the expense of separate foundations and reducing to a minimum the area of land required for conduit purposes.

The intervening space between the hot water and the cold water conduits has been advantageously utilized as a main surface water conduit to carry off the surface drainage from the Quequechan valley watershed. Embedded in the walls of the cold water conduit is a main sanitary sewer, which receives the sewage from the system of sewers mentioned above.

In the three level type of conduit here proposed, provision has thus been made for carrying down stream, in separate channels, the natural flow of the river, the storm water drainage and the sewage; while the hot water from mill condensers is carried up stream to the South Watuppa Pond. It has been found feasible to build the cold water and surface channels level from end to end, while the hot water channel has a slight slope toward the South Pond.

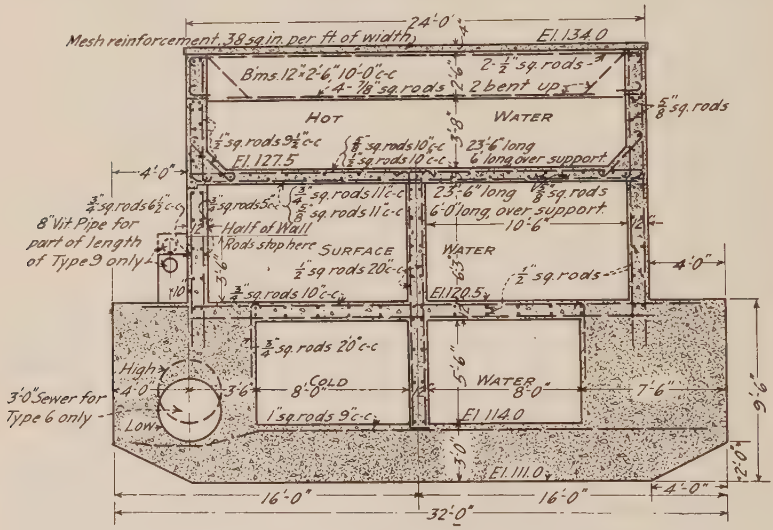

FIG. 1—SECTION OF THREE-LEVEL CONDUIT.

The incorporation of these several channels into a single concrete conduit of three-level type has resulted in a most economical and unique design.

So far as is consistent with stability, the main three-level conduit has been designed as a reinforced concrete structure. For a considerable portion of its length, however, this conduit is to be built in gravel and other porous material, in which the level of ground water will stand nearly at the top of the conduit. In view of the fact that the several conduit channels may all be empty at the same time, the lower section of the structure was designed of massive concrete to give it sufficient weight against possible flotation. Further, it will be possible, if found desirable, to deposit this mass concrete in water within the walls of the temporary cofferdam, thus sealing the bottom and rendering the unwatering of the cofferdam by pumping, a simple and easy matter. Under the program of construction proposed, however, it is

planned to dike off that portion of the river in which the conduit is to be built, and to carry out practically all the construction in the dry.

Above the mass concrete of the cold-water channel, the side walls and the floor, both of the surface-water and of the hot-water channels are of thin reinforced concrete construction. To avoid objections to an open channel, the hot-water channel is covered with thin reinforced concrete roof slabs, supported upon reinforced concrete cross-beams, the roof slabs being removable so that access to the hot-water conduit from the surface may be had at any point.

Expansion.—The usual provisions for expansion are to be made, care being taken to secure watertight joints. The hot-water channel is exposed at the surface of the ground and, because of this fact and also because it carries an intermittent flow of hot water, is subjected to greater variations of temperature than other portions of the three-level conduit; consequently provision has been made for the expansion and contraction of the hot-water channel independently of the remainder of the structure. In effect, the hot-water channel with its cross floor is simply supported on the vertical walls of the surface water channel, horizontal expansion joints being provided at the top of these walls.

The provision for longitudinal expansion of the surface water portion of the three-level conduit consists of vertical expansion joints in its vertical walls at intervals of approximately 60 ft.

No provision for expansion has been made in the mass concrete section of the cold water channel, as this is burried to a depth of about 9 ft. below the surface of the ground, the water in it is drawn from a level quite below the surface of the lake, and no considerable variation of temperature is expected in this mass concrete.

Sewer Construction in the Main Conduit.—The sanitary sewer located in the side wall of the cold water section of the main conduit is to be built as a reinforced concrete pipe sewer, in advance of the construction of the conduit itself, in order to provide for the drainage of the area of the conduit trench which is to be unwatered for the construction of the main conduit. As the construction of the conduit proceeds, this pipe sewer is embedded within it.

Cold Water Intake.—The water for the cold-water channel of the main conduit is admitted at the end of the intake in the South Watuppa Pond, about 900 ft. from shore. This part of the intake is built as a reinforced concrete box, without bottom, 18 ft. wide and 9 ft. high in inside dimensions. The box has removable covers of reinforced-concrete. The water is admitted through openings in the upper part of the sides and end of this box, the bottoms of the openings being from 4 to 5 ft. above the level of the pond bed and the tops of the openings, which are 4 ft. in height, being about $8\frac{1}{2}$ ft. below full-pond level. By the construction proposed it will be possible to draw the South Pond down 8 or 9 ft. and still maintain a full flow in the cold-water conduit; further, the water is taken at a level which will avoid the draft of any oily water from the bottom or of any warmed water from the surface.

From the box at the end of the intake to the shore, the intake structure

consists of a series of reinforced concrete sections of semicircular shape. A channel is to be dredged in the bed of the South Pond for the whole length of the intake, and covered with a thin layer of broken stone upon which the concrete intake structure will rest. The several sections of intake are to be cast on shore and taken out and lowered into position by means of scows.

Gate Houses.—Two gate houses of somewhat complicated design are provided, one near each end of the main conduit, to regulate the flow of water to and from the several channels.

Mill Connections.—Manholes are provided at the sides of the triple conduit through which water is supplied to the mill intakes. While the normal mill supply is to be taken from the cold-water conduit, two sets of gates are provided at each manhole, so that in case it is desired to empty the cold-water conduit at any time, water for the mills may be temporarily drawn instead from the surface-water conduit. The intake leading from the manhole to the mill consists of a reinforced concrete pipe, usually 36 in. in diameter, placed well under ground, at a level such that the pipe will flow full when the South Watuppa Pond is drawn down to its minimum working level.

To carry the hot-water discharge from the mill condensers back to the hot-water channel of the three-level conduit, wooden flumes have been used. These flumes must in any case be kept at high elevation, practically at the proposed surface of the ground, in order to afford gravity flow, and in most cases the ground will be filling placed upon the present flats. These cheap and flexible wooden flumes will serve admirably until such time as this filling has settled to a permanent level and permanent return channels can be built.

Branch Conduits.—Many of the mills are remote from the proposed main conduit and to supply them branch conduits have been designed. These are generally of the three-level type, and in certain cases include a sewer. In the Cornell and Stevens branches, however, it is not necessary to provide for storm water or sewage, and these two branches are of two-level type, carrying cold water and hot water only.

Grit Chambers.—To partially clarify the storm water collected by the surface water drains, grit chambers are provided wherever these drains connect with the main or branch conduits. In each case the grit chamber is a reinforced concrete structure so designed that the maximum storm flow will pass through the chamber at the rate of one foot per second. The chamber is usually 60 ft. in length. Suitable baffles are provided and it is expected that much of the solid matter washed from the streets will be deposited in the chamber during the minute of time that the storm water is passing through it.

Outfall Sewer.—The outfall sewer previously mentioned is designed to receive both the storm water and sewage flow at the lower end of the triple conduit and take it to tide water in the harbor. The sewer is to be a mile in length, about half of tunnel construction in rock and the remainder of reinforced concrete construction in open trench. Provision is made to take the sewage flow into deep water at some distance from shore, so that it may be carried off by tidal currents without offense.

Summary.

From what has been stated, it is apparent that this system will perform the functions demanded of it.

1. It will draw from the pond at low level and deliver to the mills a constant and ample supply of clean, cold water.

2. It will receive from the mills above the Watuppa Dam the water heated by their condensers and will convey it back into the pond at high level, where, after purification and cooling, it will become available for use again.

3. It will receive the surface drainage from the slopes of the valley and discharge it through the outfall sewer into the tide water.

4. It will carry away at times of freshet any surplus water accumulating in the ponds and thus obviate, for the future, any danger of floods.

5. It will receive all the sewage which it will ever be desirable to carry along this route, and will discharge it into tide water through the outfall sewer.

6. By substituting for the river the covered channels used for the above purposes, it will enable the existing flats to be filled, the nuisance to be abolished and this large amount of valuable land to be added to the business and manufacturing district of the city.

Each of the channels required must receive large and important branch channels. Were any two main channels placed side by side, one of them could not be reached by branches on one side, except by passing under, over or through the other channel, a matter involving difficulties in construction and still greater difficulties in maintenance and operation. Consequently, the main channels are placed one over the other. The result is almost of necessity the cellular type of construction.

No material heretofore known has anything like the advantages for such construction as has concrete. The massive concrete can be placed under water in the foundation, as can no other type of watertight masonry. The reinforced concrete walls and floors, acting not only as slabs but as vertical and horizontal beams or girders, provide a rigid structure which can be erected on the not always firm foundations, and can be surrounded by freshly deposited filling and subjected to the heavy earth pressures developed by it with confidence that settlement or deformation will be slight, since the structure will of itself tend to span across the soft or yielding material and deliver the pressures wherever it can best find supporting resistance. Concrete, both from the materials composing it and the conditions under which it must be placed, is exceptionally adapted for such a structure and such a locality. Reinforced concrete is the only material, whether masonry or metal, yet known which could perform these functions so economically and at the same time permanently resist the deteriorating effects of the earth and water in contact with it.

It is hoped that the city of Fall River and the mills interested will soon take advantage of this possibility and by the expenditure of something less than three millions eliminate the nuisance of the Quequechan River, greatly improve the water supply of the mills and add to its territory an attractive and valuable area, nearly 200 acres in extent, in the heart of this, the largest cotton manufacturing center in the United States.

THE CONCRETE VIADUCTS AND BRIDGES OF CINCINNATI.

By Frank L. Raschig.*

Previous to 1910, there was only one reinforced concrete bridge in Cincinnati, but that was a pioneer. This bridge is located in Eden Park, and is a Melan arch of 70-ft. span constructed in 1895. It was designed and built by F. von Emperger and has for its reinforcement 9-in. curved I-beams, spaced 3 ft. apart. The thickness at the crown is 15 in., and at the springings the thickness is 40 in. This bridge carries what is probably the earliest example of ornamental railing built entirely of Portland cement concrete in the United States.

Since 1910, the city of Cincinnati has expended about $2,000,000 for concrete bridges and viaducts. Of this amount $1,300,000 has been spent on construction work and $700,000 for property on which the various structures are built. Two viaducts have been finished; two are now under construction and will be finished early this summer. Fifteen bridges and viaducts of lesser importance have been built and about 30 small bridges to replace wooden structures have been completed. It is the settled policy of the Engineering Department of the city that only concrete bridges and viaducts be built in the future unless it is absolutely impractical to do so.

Gilbert Avenue Viaduct.—The first large concrete viaduct completed in Cincinnati was the Gilbert Avenue viaduct, Fig. 1. This structure lies in the heart of the city, being only five blocks from Fountain Square. It was built as an extension of Gilbert Avenue, a wide thoroughfare leading to Walnut Hills, a suburb of about 50,000 inhabitants.

This structure is a strictly viaduct type, consisting of short spans of column, girder and beam construction, rather than a bridge consisting of several long spans. This type was adopted in preference to the other for two principal reasons. It was thought that in the locality where the viaduct is built that arches would not show to advantage, as they would lie close to the ground and, being surrounded by buildings, the whole structure would not be seen at one time. A bridge of several arch spans appears to the best advantage when the whole structure can be viewed from a distance, giving in themselves an idea of completeness. Besides, it was not necessary, except over Eggleston Avenue, to use long spans as it would be over a stream or deep valley. The second reason for using the viaduct construction was that it was thought that the cost would be less and this was borne out by the low bids received. There was a great saving in foundation work, as the dead weight of the structure as built is about one-half the weight of an arch, and in a location like this, where piling is required, the cost was very much reduced.

The total length of the improvement is about 1200 ft., the viaduct proper being 1036 ft. long. There are short earthwork approaches at each end

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between reinforced concrete retaining walls. The viaduct proper consists in general of 30 ft. spans of reinforced concrete construction. The span over Eggleston Avenue is 100 ft. and consists of six plate girders encased in concrete. The structure is 80 ft. wide over all, with roadway 58 ft. wide and sidewalks on each side 11 ft. wide. The maximum grade is 3 per cent. The roadway is paved with wood block. Double tracks for street cars are provided, with trolley wire posts in the center of the roadway. The structure is illuminated by tungsten lamps in clusters of five lamps supported on concrete lamp-posts.

As stated before, the greatest part of the viaduct is made up of 30-ft. spans, each supported by a row of four columns. Between the columns are

FIG. 1.—GILBERT AVENUE VIADUCT, CINCINNATI.

what might be called arch girders. These girders were designed as straight, rectangular beams, no account being taken of the arch and of the slab adjacent to the beam. The roadway is supported by ten beams. The outside beams under the sidewalk are curved for appearance sake.

In the design of the various members of the reinforced concrete construction the following loadings per square foot were used: Dead loads, wood block, 17 lb.; sand cushion, 10 lb.; concrete, including steel, 150 lb. For slabs in roadway a live load of 600 lb. per sq. ft., uniformly distributed, was used; for sidewalks, 100 lb. The beams under the street car tracks were designed for two 50-ton interurban express cars, coupled together. The roadway beams were designed for a 20-ton road roller with 25 per cent added for

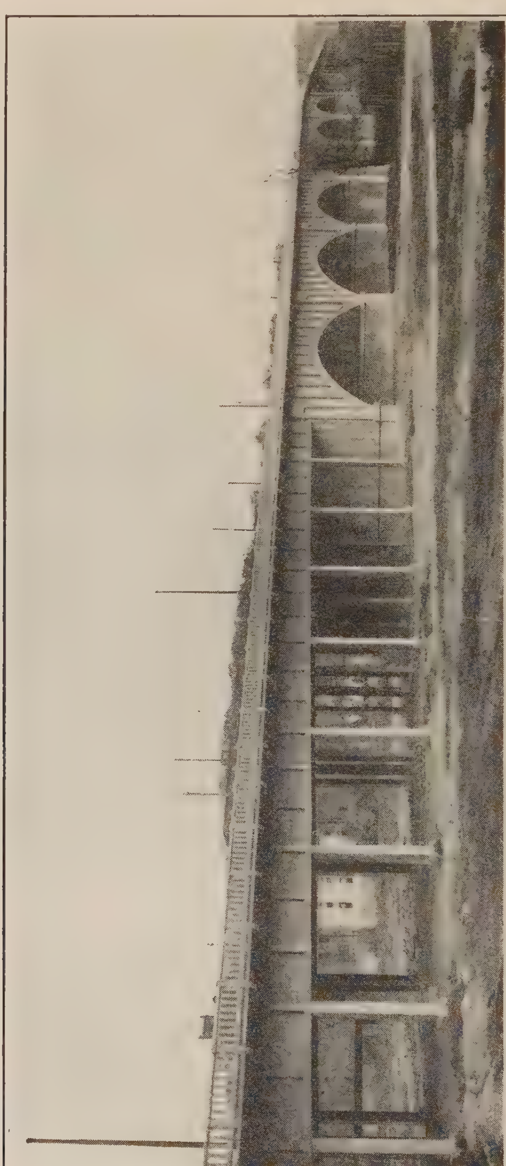

FIG. 2.—LUDLOW AVENUE VIADUCT, CINCINNATI.

impact. The following were the maximum unit stresses used in reinforced concrete design: Concrete in compression (extreme fiber stress) in slabs and beams, 600 lb. per sq. in.; in girders, 700 lb.; direct compression in inside columns, 500 lb., and in outside columns, 400 lb.; concrete in shear in the ordinary sense, 150 lb., and in diagonal tension, 65 lb. Steel in tension, 16,000 lb. per sq. in. Cold-twisted square steel bars were used throughout the work, except in the columns, where round bars were used. The concrete in the entire structure consisted of one part Portland cement, two parts sand, and four parts crushed gravel, 1 in. and under.

The ground in the vicinity of the viaduct is an old fill, probably 40 years old. The deepest portion is on the west side of Eggleston Avenue, where the depth is about 58 ft. The entire structure, with the exception of some piers at the east and west ends, rests on Simplex concrete piles, some piles being as long as 46 ft. The maximum estimated load on each pile is 27.5 tons, consisting of the dead load and the full live load. The piles were driven to sustain a load of 30 tons, using the *Engineering News* formula for computing the safe load. A test was made by loading four piles with 180 tons, the maximum settlement being ½ in. in two weeks. No difficulties were encountered in the construction of the superstructure. The usual methods employed in ordinary building construction were used. The form work was of wood supported by shores resting directly on the ground. Nearly all the concreting was done from a central plant, the concrete being distributed by steel chutes running from a wooden tower 150 ft. high. About 200 ft. of the east end of the structure was concreted by a portable plant following up the new work on the completed structure behind. A locomotive crane was used to handle the forms and steel. The steel girders over Eggleston Avenue were lifted into place by a 50-ton crane.

No surface finish was given to the concrete on the under side of the viaduct, but all the outside surfaces were finished with the cement gun. The concrete railing was sand-blasted.

The total cost of the construction work, including the paving of intersecting streets at each end, was $270,000. Property necessary for the location cost $520,000. The cost of paving the intersecting streets was $30,000, making the cost of the structure proper, complete, $240,000, or about $2.40 per sq. ft.

Ludlow Avenue Viaduct.—The Ludlow Avenue viaduct was built to eliminate a dangerous grade crossing with the Baltimore & Ohio Railroad Company's tracks. A new route was selected for the street to provide a more direct way than the old. By this means all curvature in the structure is eliminated. The old location was abandoned. To provide sufficient waterway at Mill Creek, arch spans were used over the stream.

The viaduct proper is about 1336 ft. long. This length is increased to about 1500 ft. by earth fill approaches at each end, the north approach fill being retained by concrete retaining walls. The structure is 60 ft. wide, with a 40 ft. roadway and two 10-ft. sidewalks. The entire viaduct, with the exception of the 110-ft. steel plate girder span over the railroad tracks, is of reinforced concrete, while the plate girder span is entirely encased in concrete.

A length of about 600 ft. at the middle of the structure over Mill Creek consists of six solid-barrel arches of 85 ft. clear span. The approaches at either end consist of cross girders carried on rows of reinforced concrete columns about 24 ft. apart, supporting longitudinal beams, carrying a reinforced concrete deck slab. The roadway is paved with brick and has a maximum grade of about 5 per cent.

The general architectural treatment of the viaduct was difficult on account of the heavy grades on the structure and the combination of arch and beam-and-girder spans. An appearance of uniformity was obtained by carrying along the same spandrel detail on the fascia beams of the approach spans as is used on the arch spans. While the use of false spandrel arches revealed on the fascia beams in order to make them more in harmony with the arch spans and eliminate the monotony of plain surfaces presented by the beams is somewhat unprecedented and not strictly in accordance with esthetic principles, it is believed the results warranted the departure.

In the design of the various members the following loadings were used: Dead loads: reinforced concrete, 150 lb. per cu. ft.; paving brick, 50 lb. per sq. ft.; paving sand, 20 lb. per sq. ft.; rails, connection, etc., 35 lb. per lineal ft. The live load used for approaches, roadway over arches and span over railroad was a 50-ton street car. Cars were assumed to take up 20 ft. of width of roadway. The live load on remainder of roadway on each side not occupied by cars, 150 lb. per sq. ft. Live loads on sidewalks, 80 lb. The live load for the arches proper was assumed as 200 lb. per sq. ft. for a width of 20 ft. at the center and 150 lb. on the remainder.

The following unit stresses were used in the design: Concrete in compression (extreme fiber stress) 500 lb. per sq. in.; 150 lb. in direct shear; 65 lb. in diagonal tension. Columns, concrete in direct compression, 500 lb. per sq. in. Steel in tension, 18,000 lb. per sq. in.

Cold twisted square steel bars were used throughout the work. The concrete for the entire structure consists of only one class, viz: a 1 : 2 : 4 mixture, with crushed gravel (1 in. and under) as the coarse aggregate.

The foundations for the entire structure, except that for the pier at the north side of the railroad tracks, consist of concrete piles from 25 to 35 ft. in length, calculated to sustain a maximum load of 30 tons. The pier at the railroad rests on white oak piles from 12 to 19 ft. long, with an assumed carrying capacity of 12 tons. Wood piles were used at this point because the ground was of a sliding nature which might have caused green concrete piles to be disturbed.

The approaches to the arch spans are of reinforced concrete beam-and-girder construction carried by reinforced concrete columns resting on concrete pile foundations. The arches, six in number, are placed on a skew of 52 deg., and are of the solid-barrel, three-centered type, with a clear span of 85 ft., measured parallel to the center line of the bridge, and a rise of 13 ft. 3 in. The arch rings are 2 ft. thick at the crown and are of the same thickness throughout the middle portion to points about 15 ft. from the piers, where the thickness begins to increase to 5 ft. 2 in. at the springing.

At the south end of the series of arch spans is a plate girder span over the Baltimore & Ohio Southwestern Railroad tracks. The wide right-of-way and the clearance of 22 ft. made the use of an arch span out of question. The girders in this span, 10 in number, are deck plate girders 110 ft. long. They are 6 ft. 2 in. in depth, back to back of flange angles, are all wrapped in wire mesh and encased in concrete.

The piers are of plain concrete, 10 ft. wide at the top, with a slight batter at the sides. The pier at the railroad tracks is a combination pier and abutment, taking the thrust from the arches and the vertical load from the plate girder span, and, as stated before, rests on 205 white oak piles.

In the construction of this viaduct, the gravity system of distribution was used for nearly all the concrete work. High towers were erected at such places along the viaduct as to cover all points as were worked on at one time, concrete being placed directly in the forms without rehandling. Concrete mixing plants were placed at the foot of two of the towers. One of the towers across the creek was used as a relay tower, the concrete being placed in the bucket at the foot of the tower after flowing from the tower where the mixer was placed.

The excavation for the arch pier in Mill Creek was done in a cofferdam constructed of two rows of piling, sheeted and the space between filled with blue clay. The excavation for the remaining piers and for footings of approach spans was open trench work. The concrete piles were the Raymond cast-in-place type and were driven as soon as the excavation permitted.

Arch pier forms were of wood, built the entire height of the pier and heavily braced to prevent displacement due to rapid filling of forms by the gravity chutes. As the tops of the piers were reached the stub bars to tie the arch rings to the piers were put in place. All the arch piers were completed before work was started on any of the arch rings.

The work of erecting the wooden forms for the superstructure of the approach spans followed directly behind the footing work. The form work was erected complete and the concrete placed in transverse sections, except that for the hand rail, which was erected later. The forms were built in units, so as to permit their easy removal and erection at another point farther on.

The arch ring centers consisted of wooden trusses spanning from pier to pier on timber bents resting on the pier footings and anchored to the piers. Upon these trusses, the top chords of which conformed to the curves of the arches, was laid the lagging for the arch rings. The concreting was done in longitudinal sections, one fourth the width of the arch ring. The centers were moved to the next section as soon as the concrete had set sufficiently and when the corresponding section of the next arch had been placed. After the arch centers were removed, forms were erected for the spandrel and roadway construction.

The cost of construction work, including paving and approach work, was $280,000. Property necessary for location cost $70,000. The cost per square foot, exclusive of property, was about $3.00.

Hopple Street Viaduct.—The Hopple Street viaduct is being built for

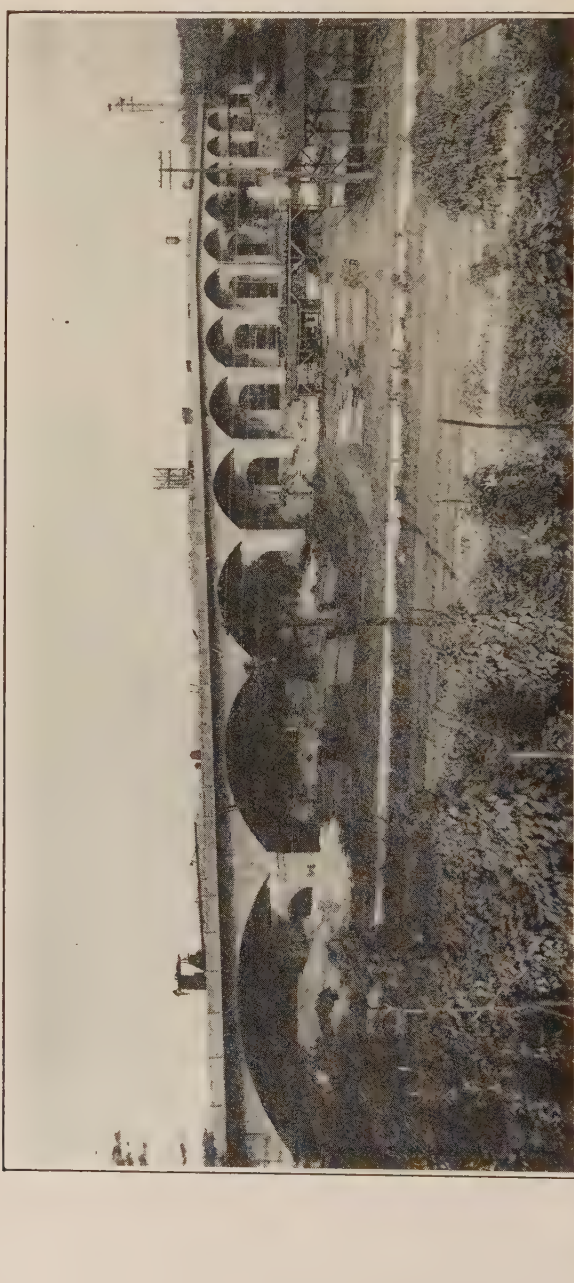

FIG. 3.—HOPPLE STREET VIADUCT, CINCINNATI.

the purpose of eliminating the grade crossing of Hopple Street and the Baltimore & Ohio Southwestern Railroad. The viaduct is placed directly over the present street and passes over Spring Grove Avenue, the Baltimore & Ohio Southwestern Railroad, Mill Creek, and the Cincinnati, Hamilton & Dayton Railroad.

This will be Cincinnati's longest viaduct. The total length will be about 2100 ft., the viaduct proper being 1930 ft. long. The structure is 60 ft. wide with a 46-ft. roadway and two 7-ft. sidewalks. The entire structure, including the railing, will be reinforced concrete, no structural steel whatever being used. The roadway is to be paved with wood block and will have double car tracks. The maximum grade is 2.7 per cent.

The bridge has the appearance of a series of concrete arches but is in reality composed of balanced cantilevers. This type of structure was adopted because longer spans could be used than in the ordinary girder spans and because it is better suited to this location where the real arch would be exceedingly costly on account of unsatisfactory foundation conditions.

A pier and the cantilever arms on each side compose a unit, the arms being balanced for dead load and for full live load. The piers are designed for bending due to the maximum eccentric load that can be applied, and considering the load on only one of the cantilever arms at a time. The piers are designed so that the pressure on the base of a pier due to this same eccentric loading will not reach an intensity greater than the unit bearing value of the soil or a greater load than 35 tons on piles. The viaduct consists of 25 skewed spans of 70, 75, and 80-ft. lengths, each span comprising two curved cantilever arms supported on reinforced concrete piers. Each cantilever arm comprises four curved ribs with a joint across the entire bridge. The joint was designed to transmit shear only.

In computing the stresses due to dead loads, the following unit weights were used: Reinforced concrete, 150 lb. per cu. ft.; wood block paving with sand cushion, 30 lb. per sq. ft.; tracks (double), 220 lb. per lineal ft.

For computing live load stresses the following loads were assumed: The two middle cantilever ribs were designed to carry two coupled 40-ton electric cars with 25 per cent added for impact. Outside ribs were designed to carry 100 lb. per sq. ft. on sidewalks and 150 lb. on the 8-ft. width of roadway adjacent to each curb. The roadway slab was designed for a live load of 600 lb. per sq. ft. and the sidewalk slabs for 100 lb. Cross beams were designed for a 20-ton road roller and 20 per cent added for impact.

The following unit work stresses were used: Tension in reinforcing steel: In piers and footings, 20,000 lb. per sq. in., and in other parts, 18,000 lb.; compression (extreme fiber) in all concrete, 700 lb. per sq. in.; shear in concrete (diagonal tension) 65 lb.; dead load per pile, 23 tons; dead load plus full live load per pile (average), 27 tons; maximum load on outside piles, 35 tons; maximum pressure on rock foundation, 8 tons per sq. ft.; on earth foundation, 3 tons.

Cold twisted square steel bars are used for reinforcement throughout and only one class of concrete, 1 : 2 : 4, using crushed gravel for the coarse aggregate.

The foundation pits were carried to different depths, and most of them required to be carefully sheathed. The material excavated was handled by two guy derricks, each derrick serving two pits. The dirt and rock was dumped into side-dump cars. Two piers, one of them in Mill Creek, were carried down in cofferdams to solid rock at a depth of about 20 ft. below water level. The cofferdams were made of U. S. steel sheet piling in 12-ft. lengths driven to rock. Other piers, which were not carried to rock, were supported on concrete piles. Raymond taper piles were used, with spirally reinforced sheet steel shells. The piles varied in length from 20 to 30 ft. A test was made of two 30-ft. piles by loading them with 90 ton of pig iron, the maximum settlement after two weeks being $\frac{5}{16}$ in.

The concrete plant which took care of the west 800 ft. of the structure consisted of a cement house with a capacity of 1500 bbl. and storage bins for stone and sand on the right and left respectively, behind the cement house, from which materials were dumped into hoppers and fed by gravity into a mixer of $\frac{3}{4}$-yd. capacity. The concrete was distributed by a steel tower 160 ft. high. An auxiliary wooden relay tower 90 ft. high, placed on the structure, was used in conjunction with this tower. This plant was dismantled and placed further east and an additional tower built to take care of the eastern portion of the structure.

The concrete piers were poured in steel forms. As soon as the piers were poured to the proper elevation, the steel forms for the 7-ft. radius semi-circular arches spanning the openings in each pier were put in place, and forms for the balance of the pier and coping were finished out in wood.

Just under the springing line of each arch in each pier two openings were left, their purpose being to receive I-beams which were passed through and extended a few feet from each face of pier to carry the centers. On these I-beams were placed heavy timbers to serve as a track on which the arch centers rested and were moved from one position to another. Blaw steel rib centers and wooden side forms were used, the two ends of the arch form being tied together by steel rods. In the case of the crossing over the Cincinnati, Hamilton & Dayton tracks, there was not sufficient clearance for the use of the rods and just before pouring, timber posts were placed at the center of the span, to support the weight. Half the width of the viaduct, including one outside rib and the adjacent inside rib from one expansion joint to the next, was placed at one pouring. As soon as the centers could be moved, they were shifted over to the other side of the viaduct and the second half poured. A locomotive crane of 10 tons capacity, running on a track alongside of the viaduct, was used for handling both wooden and steel forms and for raising reinforcing steel from the storage yard below.

A solid concrete railing of simple design is to be placed on the edge of the sidewalk and given a brush finish. The exterior surfaces of the concrete on the structure are to be sand-blasted.

The cost of the construction work will be about $420,000, making the cost about $3.30 per sq. ft.

Park Avenue Viaduct.—The Park Avenue viaduct is being built to replace an old bridge built of wrought and cast iron 50 years ago. It forms

the main entrance to Eden Park from the north, crossing a deep valley. It was therefore decided to build for the main part of the structure, an arch of long span and to make the entire work of an ornamental design on account of the location in a public park. The entire system of roadways leading up to the bridge are to be relocated, the new structure being further to the south than the old bridge.

The viaduct proper is 360 ft. long, the main portion being an arch of 180 ft. span. The arch consists of three ribs, having a rise of 31 ft. The inside rib is 10 ft. wide, the two outside ribs are 6 ft. 9 in. wide. The thickness of all the ribs at the crown is 3 ft. and at the springing 6 ft. Over the arches is an open spandrel construction consisting of the usual slab, beam and girder work. The abutments are ribbed and hollow. Leading up to the arch on each side are two 25-ft. spans of slab, beam and girder construction supported by rows of three columns.

There is a special construction over the abutments, which are 45 ft. 3 in. long. The total width of the viaduct is 65 ft., the roadway being 40 ft. wide and each sidewalk 12½ ft. wide. The roadway is to be paved with wood block and is on a 1 per cent grade. The springings of the arch are on the same level.

The following dead loads were assumed in designing: Reinforced concrete, 150 lb. per cu. ft.; wood block paving with sand cushion, 30 lb. per sq. ft.

For computing live load stresses the following loads were assumed: In designing the arch, a live load of 100 lb. per sq. ft. uniformly distributed on roadway and sidewalks; for slabs in the deck, 600 lb. per sq. ft.; for roadway beams, a 20-ton roller with 20 per cent added for impact.

The following unit stresses were used: Tension in reinforcing steel for deck and approaches 18,000 lb. per sq. in.; compression (extreme fiber) in concrete, 600 lb.; shear in concrete, direct, 150 lb., diagonal tension, 65 lb. In the arch the maximum stress in the concrete was 800 lb. per sq. in., including temperature stress and stress due to rib shortening. The ratio between the modulus of elasticity of concrete and that of the steel in the arch was taken as 20. For the footings, except those of some of the retaining walls, resting on stratified limestone, the maximum bearing per square foot allowed was 6 tons; the maximum pressure allowed on soil at the toe of the retaining walls was 2 tons per sq. ft.

A single plant is being used to mix the concrete for the entire work. A portion of the old bridge not interfering with the new bridge was left in place and under this bins were erected feeding into the mixer by gravity. A Hains gravity mixer is being used. A tower 150 ft. high is used to distribute the concrete in chutes directly to the forms. The arch rings were cast in seven sections, leaving a space of 2 ft. between each section and providing a key in the sections first cast. After all the sections were in place for several days, the keys were poured. In this way practically the entire load was on the centers before the arch was connected. Round bars 1⅝ in. in diameter, spaced 10 in. on centers, were used as the main reinforcement. The upper and lower systems were connected by ⅝-in. rods. The centering for the arches

was of the ordinary type resting directly on the ground, a maximum soi pressure of $1\frac{1}{2}$ tons being allowed for footings for centers. The centers were raised or lowered on oak wedges. Two of the ribs have been cast, the concrete being allowed to set 45 days before lowering the centers. The center

FIG. 4.—RUNNYMEDE AVENUE BRIDGE OVER WEST FORK CREEK, CINCINNATI.

FIG. 5.—BEEKMAN STREET BRIDGE OVER WEST FORK CREEK, CINCINNATI.

for the middle rib is being moved in its entirety to the west to be used for the west outside rib.

The entire outside surfaces of the structure, except mouldings, etc., are to be bushhammered. The viaduct will be completed in about six months

and will cost, with the approach work, about $135,000. The structure proper will cost about $4.00 per sq. ft.

Shorter Bridges.—Besides these four viaducts there have been completed a number of shorter bridges, a few of which will be briefly described. The bridge over West Fork Creek in Cumminsville, Fig. 4, is a cantilever type of bridge, no joint, however, being provided in the middle. The center span has a clear span of 65 ft., the total length of the bridge being 140 ft. There are two longitudinal beams connected by cross beams supporting a roadway 20 ft. wide and two sidewalks each 6 ft. wide. The bridge is on a 5 per cent grade. The total cost was about $19,000.

Another bridge on West Fork Creek further down stream, Fig. 5, is a 50-ft. girder bridge 60 ft. wide. The abutments were designed to take the thrust from the earth approach at the top and bottom, struts being built across the creek to take the thrust at the bottom. The bridge is therefore, in reality, a large box culvert. The entire structure rests on Simplex concrete piles. The cost was about $27,000.

Three of the first bridges built in the city are all of the same type, being slab, beam and girder bridges supported by columns, of a total width of 32 ft. with roadway of 20 ft. The Whittier Street bridge consists of a 40-ft. span with two 20-ft. spans on each side. The cost of this bridge was $6,500.

The Burbank Street bridge is about 300 ft. long, all the spans, except one of 40 ft. over a railroad, being 24 ft. The bridge cost about $14,000. The Powers Street bridge over West Fork Creek is 130 ft. long, consisting of one 40-ft. span and four 20-ft. spans, the total cost being about $6,000.

A bridge was built on Grand Avenue in Price Hill over a ravine to replace an old wooden trestle. This bridge is 200 ft. long, consisting of five 40-ft. spans of slab and beam construction resting on two rows of columns. The bridge is 30 ft. wide over all, with a 22-ft. roadway and one sidewalk 6 ft. wide. An elevated concrete sidewalk connects the bridge with the street to the west. The total cost was about $11,000.

During the past year the city has replaced thirty small wooden bridges with reinforced concrete bridges, mostly of the slab and beam type with iron pipe railings. Several are small arches with concrete railings. The city is planning to replace a certain number of wooden and small steel bridges each year, so as to finally have none but concrete bridges.

DISCUSSION.

In the discussion the author was asked a number of questions, the answers to which may be summed up as follows:

Mr. Raschig.

MR. RASCHIG.—Some of the reinforced concrete retaining walls were of the buttress type and others of the L type; as the patentee of one type is a resident of the city he was paid a small royalty rather than to engage in litigation over his patent. In the Gilbert Avenue viaduct an expansion joint was placed every 200 ft. from the top to the bottom of the structure, making it a number of independent 200-ft. sections. The joints were made by using tile which was broken out afterward. They were about 2 in. wide; sliding plates were placed over them in the roadway. The cement gun was not used to protect any of the steel in these bridges but simply to finish the concrete. The steel was encased in about 3 in. of concrete. Where wood paving is used a 2-in. asphalt joint was left along the curb so as to provide for the swelling of the blocks; these joints are now nearly closed. Where we finished the concrete with a bush-hammer it was to obtain the kind of surface given by that method of finishing and not because the cement gun was unsatisfactory.

CONSTRUCTION OF REINFORCED CONCRETE FACTORY BUILDING WITH SUBMERGED FOUNDATIONS UNDER SEVERE TIDAL CONDITIONS.

By N. M. Loney.*

During the winter of 1914–15 the company of which the author is the engineer decided to construct a reinforced concrete factory at Lubec, Me., in order to better protect the industry which it serves from danger of interruption by fire.

The business had previously been carried on in a frame structure supported on wood piles and, since no rail connections were available and transportation was almost entirely by boats, the building was built out over the water, the depth of which was from 8 ft. at the shore side to 30 ft. at the bay side, at high tide. The average rise and fall of the tide at this point is about 22 ft., reaching a maximum of 27 ft. at times.

Fig. 1 shows the general plan of the property. The construction shown in broken lines designates the buildings that existed on the site at the time the construction of new reinforced concrete building was undertaken. The full lines show the new three-story reinforced concrete factory building, approximately 80 x 148 ft. in plan, together with a 20 x 148 ft. reinforced concrete driveway and new boiler and engine house which, together with some other minor changes, constituted the proposed improvement.

Fig. 2 is a photograph showing portion of the site of new building. This photograph was taken during the construction of the false work and shows the condition of the beach at normal low tide.

Fig. 3 is a view taken during the construction of the building. This, together with the high-tide lines indicated on the piles of the old building, in Fig. 2, will serve to give some idea of the variation in water level.

The building was built against a narrow wagon road, back of which is a rock cliff averaging about 50 ft. high, Fig. 4. The beach was overlaid with boulders and talus from the cliff, in pieces ranging in size from a few inches up to 2 ft. in diameter, from a depth of 2 ft. near the shore to a depth of 3 to 13 ft. at the bay side. Underneath this was a layer of about a foot of cementitious gravel over the solid rock ledge, the surface of which sloped more or less uniformly in the general direction shown in Fig. 5.

The piles of the wooden bents under the original building are supported mainly on concrete footings, bearing on the talus, except the outer rows of piles, which were driven. All were strongly diagonally braced in both directions.

It had been anticipated, for some time, that a fireproof structure would be necessary as soon as conditions permitted. Two years before a reinforced

* Chief Engineer, American Can Company.

concrete gas producer house was built in the location, shown in Fig. 1, so as to fit into the proposed future development.

At that time, advantage was taken of this to try out the practicability of supporting the structure on long unbraced reinforced concrete columns, with a means of protecting them against damage by salt water and the very

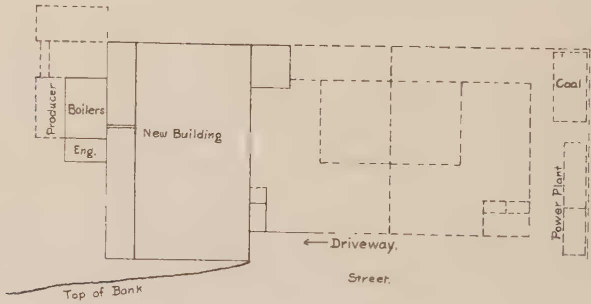

FIG. 1.—GENERAL PLAN OF THE PROPERTY.

FIG. 2.—PORTION OF THE SITE OF NEW BUILDING.

low temperature prevailing in winter. In the new building, the maximum unbraced length of columns supporting the first floor is 34 ft.

Some previous concrete work, on the site, had suffered rather severely, due to the very large variations in the water level. This small building, which was designed for a floor load of 2000 lb. to the square foot, was, there-

fore, supported on 24 in. spirally reinforced concrete columns, for which glazed tile sewer pipe, with packed and grouted joints, was used as forms, these being braced to the falsework while being concreted.

FIG. 3.—VIEW TAKEN DURING CONSTRUCTION OF THE BUILDING.

FIG. 4.—COMMERCIAL STREET BETWEEN FACTORY AND CLIFF.

This method failed to properly protect the concrete, due to leakage at the joints, and it was then decided to adopt the protection offered by means of wood kept in contact with the concrete.

The columns were repaired with wire-mesh-reinforced cement mortar and unmatched staves were fitted to them. The results obtained checked the experience of others that a wood covering, between high and low tide, formed the best protection for concrete exposed to the action of sea water and frost, and this method was applied to the new building, except that an improvement was made by using, for forms, standard wood stave pipes hooped with extra heavy hoops and heavily coated with a tar preservative. The hoops were made with round lug joints, instead of being spirally wound, to allow for replacement.

A waterproof joint was made with the footing and the pipe was erected with male and female joints so that the forms were practically dry at all times.

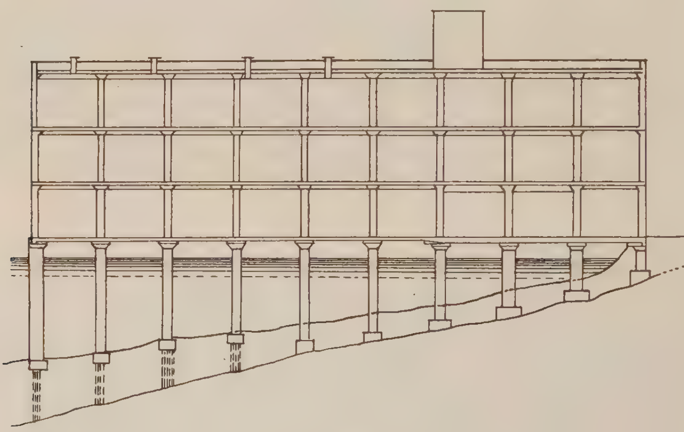

FIG. 5.—FOUNDATION CONDITIONS FOR NEW BUILDING.

The depth of rock was ascertained by sounding with iron bars, and the footings for all, except the four outer rows of piers, were carried to solid rock.

On account of the wash of the tide and nature of the excavation, heavy sheeting was required and since excavation could only be carried on at low tide periods, heavy pumping was necessary, the drainage from the upper slopes being especially troublesome.

The four outer rows of footings were carried on spruce piles driven to rock by a floating drop hammer. A number of piles were lost, due to diversion and breakage from encountering boulders in driving. Additional piles were driven in such cases and the size of footings increased and reinforced accordingly.

After driving, these piers were first coffer-dammed with interlocking sheet steel piling, which, however, failed to prove sufficiently watertight, and an outer row of matched wood sheet piling was driven and the space between filled with clay in bags. The coffer-dams were made with a working chamber at one end to allow cutting off piles.

The tops of the coffer-dams were about 8 ft. above low tide and, as soon as the tide had fallen sufficiently, the pier was pumped out with a 6-in. pulsometer, and the excavation to cut-off, the cutting off of the piles and the con-

FIG. 6.—VIEW OF A COFFERDAM.

FIG. 7.—READY TO CUT OFF PILES.

creting were all done in the dry, one operation usually being accomplished in each low-tide period.

Gravel and sand were procured from nearby beaches and transported to

the site in small sailing vessels and fishing smacks and hoisted to the overhead bins in buckets.

Both tension and compression tests were made of briquettes made up of the different aggregates available; also of both washed and unwashed samples. It was found that the strength of the briquettes made from the washed samples was no greater than from the unwashed.

Briquettes made from certain local sands showed 170 per cent of the tensile strength of briquettes made with standard Ottawa sand, but compression tests of briquettes made with the same sand did not show as great strength as certain other and more available sands, which only showed up equal to Ottawa sand on the tensile tests. This indicates that it is not advisable to depend on tension tests only when selecting sand for concrete.

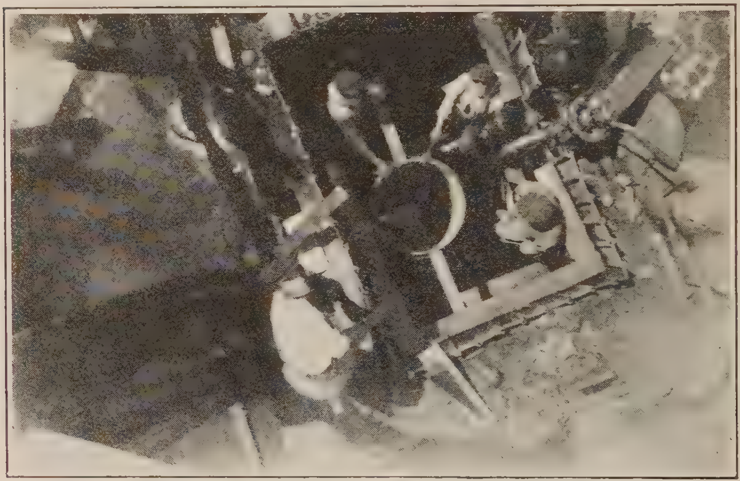

FIG. 8.—VIEW SHOWING THE FIRST SECTION OF COLUMN FORMS BEING PLACED.

On account of the lack of space it was necessary to construct temporary platforms over the beach for the storage and fabrication of material. The forms for the first floor were built on a previously designed system of pile supports, similar to the construction under the old building and, on account of the probability of heavy storms, this was made very strong and rigid and thoroughly braced. The forms were of 2-in. plank supported on a joist construction. The piles and bracing of the falsework were so located that they allowed the placing of the forms for the concrete columns, caps and floor depressions without interference.

These forms were used for a working platform for the mixers, derricks and other equipment used in placing the concrete in the footers and columns. The forms next to the shore were first constructed and the equipment moved out as the work progressed, so as to enable the derricks to work over the edge

of the platform for the placing of the material. For the placing of the outer rows, where the time the tide was out was very short, the concrete was handled in large buckets. For the inshore sections chutes were used.

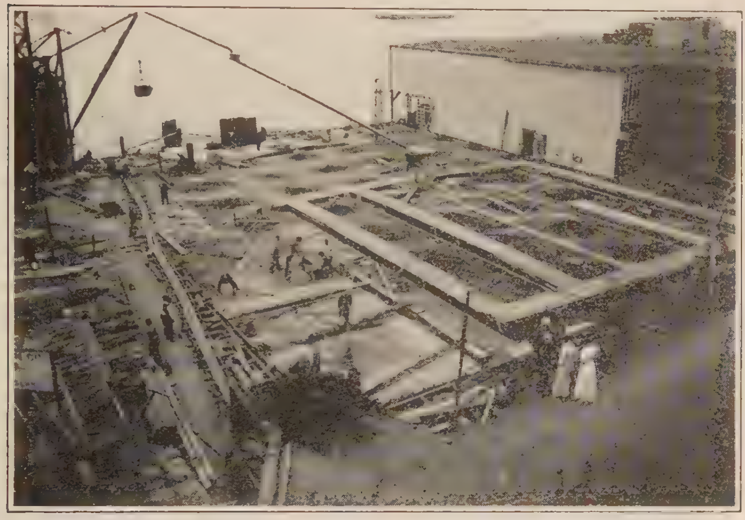

FIG. 9.—GENERAL VIEW OF FIRST FLOOR.

FIG. 10.—PLANT OF AMERICAN CAN COMPANY AT LUBEC, ME.

The joints of the column forms with the footings were first made by setting the forms in a recess in the footer packed with asphalt, the form being fastened down with angle brackets by bolts previously set in the concrete.

The bolts also served to overcome the buoyancy of the forms. It was later found more effectual and convenient to set the bottom section of the forms a few inches into the concrete, when the latter was freshly poured, temporarily anchoring them to the falsework.

Fig. 6 is a view of a coffer-dam for one of the outermost row of piers. This method of constructing column forms allowed the concreting of the columns to proceed regardless of tide conditions, as well as protecting the concrete while setting.

The forms have not been removed for inspection of the concrete, since extra care was used in pouring. The concrete mixing for the first floor and the balance of the super-structure was performed on a working platform erected between the coal dock and power plant, in such a location as to allow for convenient unloading of material.

The floors of the main structure are constructed on the flat slab system with a four-way reinforcement. That portion of the first floor which was designed for 1200 lb. per square foot is now loaded to 1800 lb. to the square foot without showing appreciable deflection. The power plant is of beam and girder construction, especially designed for the power equipment which was placed on it.

Fig. 7 is a close view of same, ready to cut-off piles; Fig. 8 is a close view showing the first section of column forms being placed.

Fig. 9 is a general view of first floor while being concreted; Fig. 10 shows the completed building.

The building was constructed by the H. P. Cummings Construction Co., of Ware, Mass., under the general supervision of their Mr. H. Bell. The owner's interests were under the general charge of the writer, who designed the structure, and under the inspection of Mr. W. Nopper, who had considerable previous experience in concrete work in the vicinity.

A great deal of credit is due the contractors for the quality of the work obtained under difficult circumstances. The labor conditions were especially poor, since the majority of the workmen had to be imported and housed by the contractors in construction camps and, in addition, the work had to be carried on day or night whenever the tide permitted.

CONSTRUCTION OF THE AUSTIN, TEXAS, RESERVOIR AND DAM.

By Lamar Lyndon* and Frank S. Taylor.†

In presenting this paper, it is understood the usual standard methods of concrete work are fully known to the members of the American Concrete Institute, and it therefore is useless for us to dwell on any of the ordinary features of design and construction. The object of this paper is to set forth a few of the unusual characteristics of the work and point out particularly features of design which did not prove satisfactory in practice.

The Austin Reservoir.

The Austin Reservoir is a circular reinforced concrete structure averaging 12 ft. in depth, 392 ft. in outside diameter, and it has a capacity of 10,000,000 gal. It is divided into four segmental sections by means of two walls, each of which runs across a diameter of the reservoir, these two walls being at right angles to each other and intersecting at the center of the structure. The lower 6 ft. of wall is inclined inwardly, the inclination being 5 ft. in the height of 6 ft. This lower section of wall is supported on a series of external buttresses spaced 12 ft. apart, and the lower wall section spans from buttress to buttress as a simple beam, the thickness of the concrete and the area of the reinforcing steel being computed for simple beam action. The upper section of the wall is vertical, and this is unsupported externally. It acts as a cantilever and is so designed. A resultant of all the forces acting on any section cuts the base of the buttresses at a point about 4 ft. 6 in. from the outer end, and 6 ft. 8 in. from the inner end. This means that in order to equalize the foundation pressures, the buttress footings are spread near the outer end. The cross division wall is simply a wall battered on both sides and reinforced as a cantilever. When one division of the reservoir is emptied, this wall must, of course, withstand the water pressure due to 12 ft. depth against it, and as the divisions are alternately filled and emptied this wall must be able to resist a pressure applied on either side.

Expansion Joints.—Two expansion joints were placed in each of the cross walls, there being one joint midway between the rim of the reservoir and its center in each wall. Also, there were four expansion joints placed in the outer rim of the reservoir, each joint being midway between the points where the cross walls joined the rim. Although the circumference of this structure is about 1200 ft., it was assumed by the engineers that practically no provision was necessary to take care of the expansion of the rim, because of its circular section, and these four expansion joints, dividing

* Consulting Engineer.
† Resident Engineer.

it into 300-ft. sections, were merely placed as a factor of safety. It was found that the assumption that the circular ring would expand and contract without cracking, was incorrect, and, in view of the fact that much of the concrete was cast in the summer, there should have been double the number of expansion joints placed that were actually used. The first winter after the construction of this reservoir, cracks appeared at numerous places in the rim, and were not effectually stopped until they were cut out with a cape chisel and grouted together during the cold weather. The expansion joints did not leak at all, and the change in length of the walls, with change in temperature, was clearly visible at the expansion joints.

The expansion joints were made by placing an 8 x $\frac{1}{2}$-in. strip of steel plate in the form and bulk-heading it off, so that when the form was poured full the wall ended at the expansion joint; the concrete adhered firmly to the steel strip, of which 4 in. was inserted in the form and 4 in. left projecting over the edge of the finished section. After stripping the form, the edge of the concrete and the projecting portion of the steel plate were thickly coated with asphalt, and the next section of concrete cast; thus leaving the two abutting ends of the concrete disconnected, and the steel plate acting like a tongue working in a co-acting groove.

In order to maintain the monolithic nature of all work on the walls, between expansion joints, it was necessary to pour one-half of each cross wall together with one-fourth of the circumference wall, without interruption. This was accomplished by pouring from scaffolding above the final height of the wall. This scaffolding provided for the runways of concrete cars. Concrete was taken into the cars from an elevated hopper, which was served by a hoist situated above the mixer, near the center of the reservoir in one of the compartments.

Form Work.—Forms were provided for one-fourth of the entire construction. The forms to be placed for the inclined beams were supported by lugs held by bolts through the buttresses. These forms were afterwards struck by simply removing the lugs and allowing the forms to fall out. The inside forms for the beam section were braced from posts set firmly in the ground at the bottom; the top was held by bolts through pipe separators to the under form. The outside form of the upper section was placed before concreting was begun. The concrete was run periodically over the entire horizontal area of the beam section of the form until it had reached within a few inches of the height of this section. The upper forms were placed while concrete was being run in other parts of the same monolithic section, so that a continuous and monolithic section was obtained from the foundation to the top of the reservoir between expansion joints.

Concrete.—The thickness of the wall varies from 6 to 17 in., and the floor slab is 4 in. It was necessary to provide a concrete that had maximum density in order to assure an impermeable structure. This was obtained by carefully measuring the voids in the aggregate while the concrete was being mixed and the voids always filled with cement. It was provided in the estimate of cement necessary to fill the voids that an amount of hydrated lime equal to 8 per cent of the weight of the cement would be used, and that

the total volume of the cement and lime would be equal to the voids shown in the aggregate, less 9 per cent of the volume of aggregate allowed for water of crystallization, plus 4 per cent for increased volume due to the further separation of the particles of aggregate by the cement and lime and for imperfect mixing. The separation of the particles of aggregate by the cement was found to be usually less than 1 per cent, but the uniformity of distribution of cement throughout the voids in the aggregate was less certain, and our experience required a safety factor for this item of 4 per cent in the quantity of cement necessary to provide for the voids as tested.

Floor.—The floor or pavement was laid directly on the natural surface, after it had been cleared off and about 12 in. of loose material removed. Before laying the pavement, a 5-ton road roller was moved over it, so that the unit pressures to which the earth was subjected were greatly in excess of those which could be imposed by the weight of the water in the reservoir. This pavement was 4 in. in thickness, except at the expansion joints, where it was made 8 in. thick, the expansion joints being simply overlapping joints, such as are well known in the art.

It was decided, after some experimentation, that expansion joints in the floor slabs could be placed as much as 50 ft. apart with safety, so that the floor or pavement of the reservoir was divided into slabs 50 ft. square. The expansion joints were made by the forms over the part of the joint which projected under the next slab. After these forms were struck, it became necessary to provide an elastic material against the under lip of the expansion joint so that the next concrete poured would not be against that lip by whatever thickness we estimated the expansion of the slab would require. This distance was found to be about $\frac{1}{2}$ in., and the lower lip was provided for by rolling tarred paper to a sufficient thickness to make it the required $\frac{1}{2}$ in. when flattened. This was rolled against the lower lip, and the upper lips were separated by a piece of lumber $\frac{1}{2}$ in. thick. The face of the expansion joint on the first slab run was painted with asphalt before the tarred paper or lumber strip was placed.

Before concreting the pavement, 2 in. of sand was placed over the surface to be covered by the pavement, thoroughly wet, and on this sand the reinforcing material, No. 12 4-in. triangular mesh, was sewed together and spread over the entire surface of the slab to be concreted. The concrete was mixed and poured in strips 4 ft. wide, the contact of new concrete with old being separated by a period of time not longer than 12 minutes. As soon as one run of concrete was finished and the next one begun, a man was provided to follow the run to pull up the reinforcing material into the concrete. This was done by a steel hook about 12 in. long, which the operator thrust into the concrete until he had encountered the reinforcing, after which he pulled it up into approximately the middle of the 4-in. slab. After the concrete had set from two to four hours, depending upon the temperature, other operators with long-handled floats smoothed the surface of the concrete to an approximate plane. This operation proved successful, and, after two years' use, no cracks, or even checks, due to temperature changes, have appeared in any of this pavement.

The New Austin Dam.

The dam which was built to replace the original dam carried away by a flood about sixteen years ago, is a hollow reinforced concrete structure. The buttress walls are spaced 20 ft. apart, and in addition to these, longitudinal walls were built which lean diagonally upstream and are perpendicular to the buttress walls; the deck slabs rest on these two sets of interlocking, supporting walls. In this way the buttress walls are braced against lateral stresses, and the section of the individual deck panels, which take the entire water pressure, is a square. This admits of reinforcement in two directions, and it was so placed, the spacing being logarithmic, the distance apart of the bars growing less as the center of the panel is approached.

Owing to the difficult foundation conditions, it was desired to make the length through the base, from upstream to downstream, as short as possible, at the same time the usual parabolic spillway curve had to be preserved and the angle of the deck kept within standard practice, namely, 42 deg. to the horizontal. These two conditions, with the height of dam, usually fix the length through the base. In this case, however, a design was made of a contracted section in which the spillway crest starts some 22 ft. upstream from the end of the inclined deck, and in this way the base length is reduced by approximately this length. It was found in practice that the saving in material, by using the square slabs supported on all four sides and the contracted spillway, was considerable, but the cost of form work was largely increased, so that, although there was a net saving, it was not so great as the value of the reduced quantity of materials. The panels were all treated as continuous beams except on those sections where expansion joints were placed, and these were computed as simple beams.

Automatic crest gates were placed on top of the dam between piers, which piers were merely continuations upward of the buttress walls. On top of these piers a reinforced concrete bridge was constructed, and on this a railway track was placed. The expansion joints used were identical with those which were used in the construction of the reservoir and which have before been described. These were placed at intervals of 100 ft., every fifth panel being an expansion panel.

The total length of the dam, when completed, was 1500 ft. on the crest. Of this 300 ft. comprises a concrete core wall ranging from 60 to 90 ft. in height and sunk in a trench of from 3 to 6 ft. greater depth than the wall height at any point; 120 ft. is a bulkhead section of masonry, 540 ft. is the reinforced concrete spillway and 540 ft. is solid masonry spillway, which was made of the remaining section of the old dam that still stood in thoroughly good condition.

The height from foundation to top of bridge varies from 74 to 90 ft. The net elevation in water level is 65 ft. above the stream bed.

Foundation Site.—The failure of the old dam in 1900 was due entirely to bad foundation conditions. With this in mind, the engineers took particular care in preparing the foundation for the new section. The stream bed is

wholly of limestone, which is cracked and fissured and has numerous cavities. All the trenches for the buttress walls, the cross-wall footings and the vertical cut-off walls at the upstream side of the dam were sunk down to a thoroughly good rock and the cut-off wall was carried below practically all permeable strata. In the bottoms of these trenches, drill holes were made varying from 6 to 18 ft. in depth. These were carried down until they had penetrated 5 ft. or more into solid rock. Compressed air was then applied to the hole, and, where they connected with cavities or fissures, they were blown out clean, and in some cases, quantities of thick mud were squirted out through adjacent holes by the action of the compressed air at 80 lb. pressure per sq. in. Grout was afterwards forced into the drill holes by means of an ordinary grouting machine, and in many instances grout forced into one hole would finally begin to emerge from some other hole 40 or 50 ft. away. The final pressure on the grout was 80 lb. to the sq. in., and it is believed that every crack and seam was thoroughly sealed. After the grout had hardened, intermediate drill holes were made and the solidity of the bottom proven by the character of penetration of the drill. This work required care, skill and considerable patience. In some instances extraordinary quantities of grout would be required to fill the cavities with which the drill holes connected, and as much as 7 cu. yd. have been forced into one 2 in. drill hole.

Turbine Chambers.—Reinforced concrete has been used for setting vertical turbines in scroll-shaped cases, the draft tube being also of concrete and forming an extension from the turbine chamber. It is believed, however, that this is the first instalation where a pair of vertical turbines has been set in a cylindrical case of reinforced concrete under a comparatively high head, 65 ft., with the water led to the cases through a long steel penstock and discharged through a steel plate draft tube. These cases were made of somewhat larger diameter than is usual when they are made of steel plate. They are slightly oval in cross-section instead of circular, so that the turbine gate shaft could set inside the cases.

The chamber covers are heavy cast iron disks, dished and ribbed to withstand the water pressure. These are made in half sections so as not to be too heavy and cumbersome in erecting. Each one is held in place by machine screws which bolt it onto a heavy cast iron ring, the latter being set into the upper surface of the concrete of the chamber, and it in turn is held down by 32 bolts, each $1\frac{1}{2}$ in. in diameter, which extend approximately 3 ft. down into the concrete. The reinforcement is steel hoops at the top and bottom of the chamber, the intermediate portion being carried by vertical rods which are placed to resist simple beam flexure. As there was some apprehension that these cases might crack, there was an excessive amount of reinforcing steel placed in them, and the maximum tensile stress limited 10,000 lb. to the sq. in.

Waterproofing.—The concrete used in making the deck slabs of the dam and the turbine chambers was all waterproofed in the same manner as that of the reservoir. Before adopting this method of waterproofing, numerous tests were conducted by making up sample blocks of concrete with various

percentages of hydrated lime and subjecting them to a water pressure of 80 lb. per sq. in. for several months, and the adoption of this percentage was the result of these experiments. Other tests were made on the tensile and crushing strength of the concrete and the change in these as compared with the strength of unwaterproofed concrete was found to be so small as to be practically negligible.

CONSTRUCTION OF KENSICO DAM.

By Wilson Fitch Smith.*

The Kensico Dam of the Catskill Water Supply System for the City of New York is an engineering work of considerable interest, not only on account of its size and the remarkable progress that was made in its construction, but because it is the first large dam in this country to receive serious architectural treatment.

The Kensico Dam is located in the upper valley of the Bronx River in Westchester County, 3 miles north of White Plains, and forms a reservoir covering 2218 acres. It will serve as a storage reservoir near the city for the water collected in the impounding reservoirs of the Catskill Mountains. Its available capacity of 29,000,000,000 gal. will afford nearly two months' supply at maximum draft, permitting at any time closing down for inspection, and repairs if necessary, the 75 miles of aqueduct between it and the Catskills.

Features of Design.—The dam is a straight, gravity type, 1825 ft. long on top, 28 ft. thick under the coping, and 307 ft. high from the lowest point in its foundation to the roadway on top, which is at El. 370 above mean sea level, 15 ft. above the full reservoir flow line. It is built of cyclopean masonry. The water side is faced with concrete blocks, and the exposed downstream face is of granite masonry. It contains about 900,000 cu. yd. of masonry.

The special features of its design are the division into 23 sections by transverse contraction joints placed 79 ft. apart, the inspection wells on these joints near the upstream face, the water-stops in the contraction joints formed by strips of sheet copper inserted in the joints after the masonry was completed, and the system of porous concrete drainage wells near the upstream face.

Observations of many continuous masonry dams in this latitude indicated that transverse shrinkage cracks are sure to occur, and while these are of no serious moment to the stability of the dam, they permit the seepage of water to the exposed face, resulting in unsightly disfigurements which, to the lay observer, suggest evidence of poor workmanship if not serious structural defects. By providing definite transverse joints at such intervals as experience suggested, the shrinkage in Kensico Dam is concentrated at these points, and definite provision against leakage through these joints is made by the water-stops and inspection wells. The transverse joints are in the form of vertical tongues and grooves, each 10 ft. wide and 1 ft. deep, extending from near the foundation to the top of the dam. An inspection gallery extending the entire length of the dam just under the roadway gives access to the top of each inspection well, and a lower gallery following the profile of the refill connects with the lower portions of the wells. The porous drainage wells, spaced 15 ft. apart along the length of the dam, extend from the upper to the lower gallery and are expected to intercept any slight seepage which may

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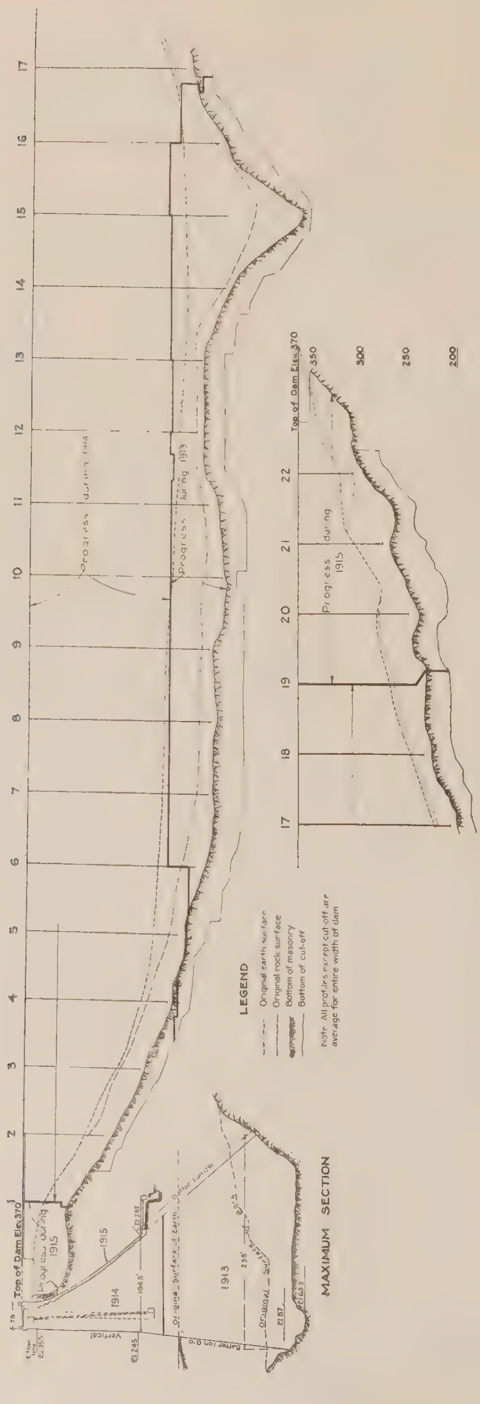

PROGRESS DIAGRAM OF THE CYCLOPEAN MASONRY AND CUT-OFF, KENSICO DAM.

occur through the face of the dam and conduct it to the lower gallery and thence to a drain connecting with the blow-off conduit.

Kensico Dam is the most impressive structure of the entire Catskill System. Located at the northern terminus of the Bronx River Parkway, with the Harlem Railroad on one side and an important State highway on the other, it is readily accessible from the city and will be visited by thousands as one of the city's most interesting works. For these reasons it was accorded architectural treatment in keeping with its monumental character. The broad expanse of its downstream side is faced with stone masonry of suitable design, and the approach is graded and laid out with drives and planted areas, the whole forming a complete and dignified treatment. (Fig. 1.) The visible base of the dam is a broad terrace nearly 1000 ft. long, extending across the level floor of the valley, the ends of which are marked by cut stone pavilions and broad flights of steps descending to the paths leading to the concourse. Above the terrace, the face of the dam, 130 ft. high, is divided into panels corresponding with the sections between contraction joints. Each panel is separated from the adjoining one by a band of heavily rusticated dimension stone rising from the terrace to the cornice. Within a fine-pointed border, the field of each panel is of roughly squared masonry, from which project dimension-stone headers arranged in diaper pattern. Beyond the ends of the terrace, which are marked by double rusticated bands forming pylons, the panels are continued up each hillside in symmetrical arrangement. The main feature of the treatment is the heavy cornice, which extends the entire length of the dam, and is flanked by circular cut stone pavilions of classic design, through which the roadway passes. The cornice includes a roughly carved frieze $6\frac{1}{2}$ ft. high, suggesting the shield and wreath design. This is surmounted by cavetto and torus courses, the latter being 4 ft. high. A stone parapet completes the top of the dam.

The space between the terrace and concourse will be occupied by a large pool, fountains and formal planting. The approach to the dam is a gradually descending plane nearly 2000 ft. long traversed by broad avenues. The Bronx River Parkway, which follows the Bronx River from New York City, will connect with these avenues.

The dam and reservoir were built for the city of New York by H. S. Kerbaugh, Inc., under a contract, the amount of which, based on the quantities of the preliminary estimate, is $7,953,050.

Methods of Construction.—The purpose of this paper is to describe the methods of construction employed by the contractor in building the dam, which enabled remarkably rapid progress to be made in the placing of masonry. The work was begun in 1910, and the first eighteen months were devoted to building a substitute supply for the old reservoir, which had to be drawn off to make the dam site available. Excavation began in the fall of 1911, and involved the removal of about 600,000 cu. yd. of earth and rock. The rock on the hillsides had only a few feet of earth cover, but in the floor of the valley it was about 30 ft. below the surface, and at one place a preglacial gorge existed where the rock was about 120 ft. below the valley floor. The excavation was made by steam shovels and removed in 8-cu. yd. car trains on

FIG. 1.—PERSPECTIVE VIEW OF KENSICO DAM, SHOWING THE ARCHITECTURAL AND LANDSCAPE FEATURES.

standard-gage tracks. The excavated earth and rock from the deeper part of the gorge were removed by cableways.

The excavation was prosecuted so vigorously that by the spring of 1913

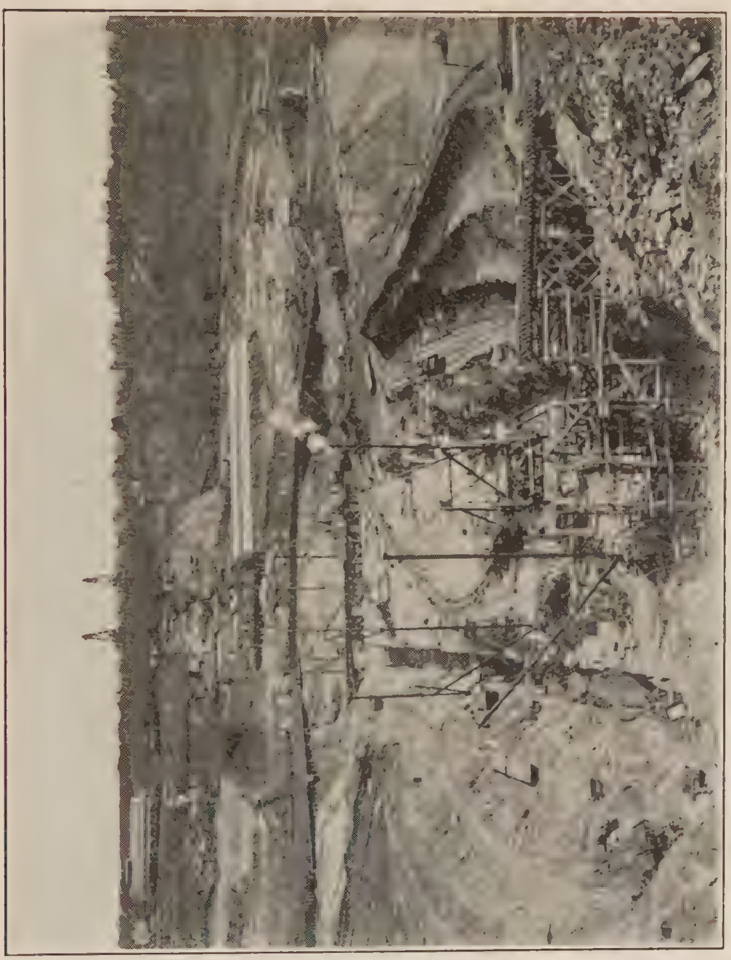

FIG. 2.*—KENSICO DAM SITE, LOOKING EAST FROM WEST HILL.

the gorge and a considerable portion of the broad area between the gorge and the west hill were ready for masonry. The method chosen by the con-

* This illustration shows the foundation nearly ready for concrete and the excavation for the cut-off trench in progress. The trestle in the foreground is being erected for the concrete mixing plant, and just beyond the first-lift traveler is being erected. In the middle distance, the trestle crossing the excavation is on the west side of the gorge. The flume on the east side of the gorge carries the local drainage past the dam site. View taken March 28, 1913.

tractor for delivering and placing the masonry was somewhat of a departure from the recent American practice on large masonry dams. The use of cableways for delivering the materials to the placing derricks was discarded on account of the limited speed of cableways. Instead all the materials were

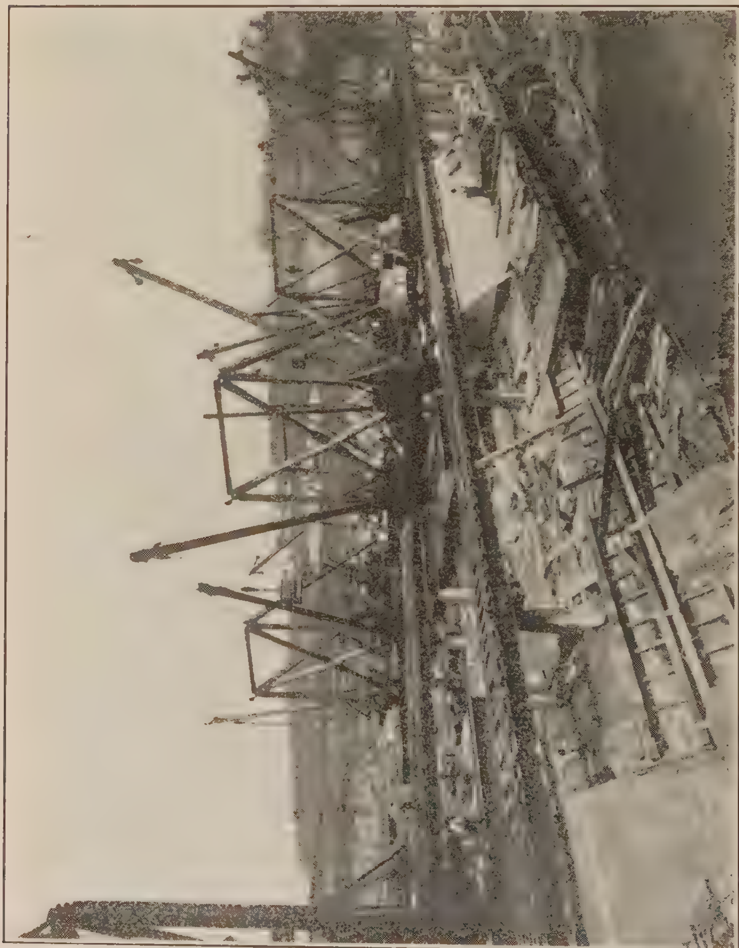

FIG. 3.*—CONSTRUCTION OF SECTION 7, FIRST LIFT.

delivered directly to the placing derricks by rail. (Fig. 2.) Delivery tracks of standard gage were provided on berms in the excavation slopes on both sides of the dam, and on the dam itself, so that the speed of building was

* This view shows the travelers, traveler track removed over the section, service track and downstream face forms. View taken July 28, 1913.

limited only by the capacity of the derricks. The method also took advantage of the structural feature of the division of the dam into sections by the contraction joints, which enabled any section to be built to any desired height

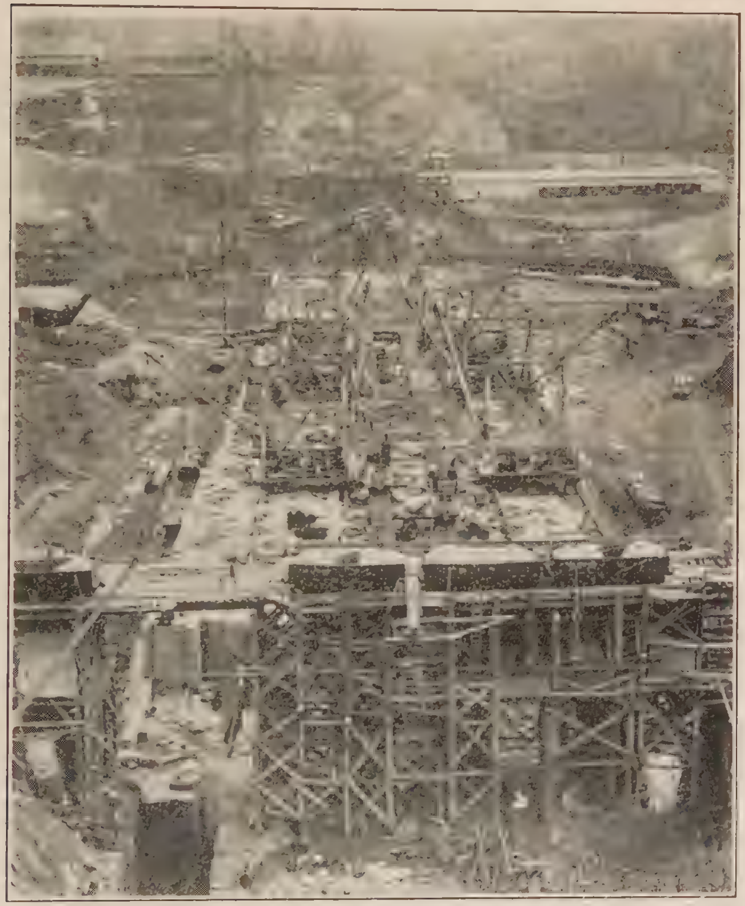

FIG. 4.—GENERAL VIEW FROM WEST HILL TOWARD EAST HILL.

In the foreground are the concrete plants for the travelers, service tracks, shuttle cars and travelers constructing the second lift of the masonry. View taken October 30, 1913.

independently of the remainder of the dam. The plan for the main portion of the dam was to build one section at a time about 25 ft. high and then proceed to the next section, bringing it to the same height. After a considerable portion of the dam had been built to the height of this first lift, the operation

was repeated for the second lift of the same height, and so on to the top of the dam.

As it was expected to handle cyclopean stone of the largest practicable

FIG. 5.—VIEW OF GORGE LOOKING NORTH FROM DOWNSTREAM SIDE OF DAM.

View taken May 14, 1913.

size, derricks of large capacity were provided. The derricks, instead of being single units resting on the masonry, were mounted in pairs on travelers, and these travelers were carried on elevated tracks parallel with the axis of the dam, supported on concrete piers rising from the foundation or the completed

masonry. This arrangement made it a simple matter to move the derricks from section to section until the lift was completed, when they were raised to new tracks for the next lift.

The travelers were rectangular timber frame platforms 30 ft. wide by

FIG. 6.*—VIEW LOOKING WESTWARD FROM BASE OF EAST HILL.

39 ft. long, about half the length of a section. (Fig. 3.) The derricks were mounted at the two corners of one end. The masts were cross-braced and

* In the foreground are the derricks for placing masonry in the gorge, the trestle and mixing plants on the west side of the gorge, and in the distance the elevated travelers, service tracks and berm tracks. View taken July 29, 1913.

the stiff legs carried to the back ends of the platforms. The masts were 30 ft. high and the booms 57 ft. long and could operate through 270 deg. of horizontal arc. Each derrick was equipped with an electric, double-speed

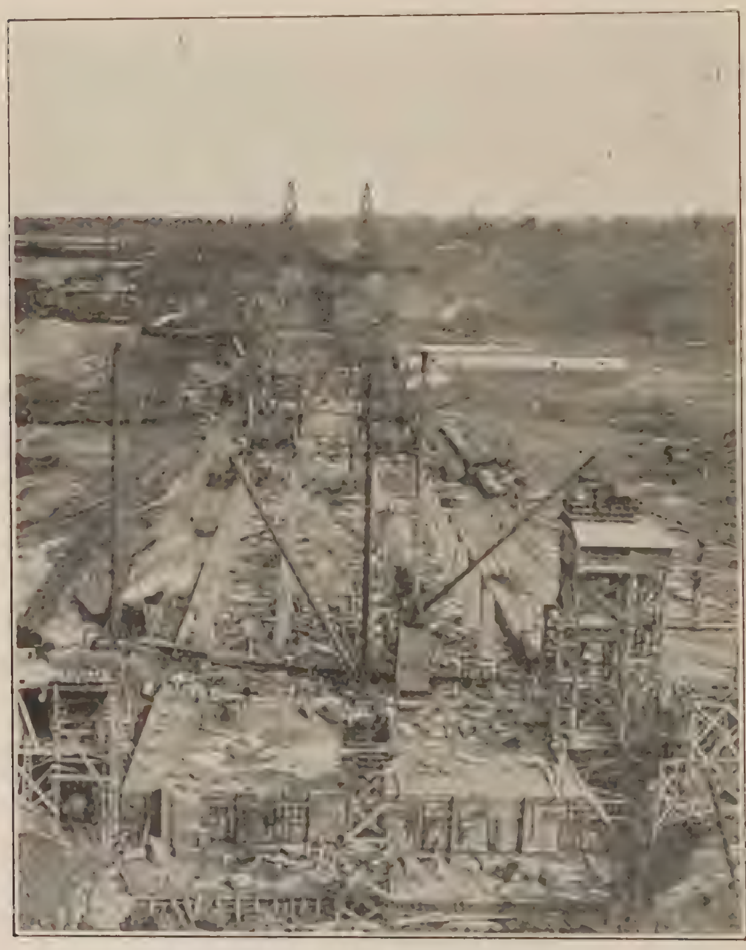

FIG. 7.—GENERAL VIEW FROM THE WEST HILL.

This shows the arrangement of the mixing plants and service tracks at the beginning of the second season. The joint in the foreground shows the arrangement of concrete blocks to form vertical tongues and grooves. View taken April 8, 1914.

hoist of 75 h. p., carried on the platform. Fully equipped with counter-weights, each traveler weighed about 100 tons.

The rails for these travelers were carried by longitudinal stringers of

28-in. 80-lb. Bethlehem H-beams, 39.5 ft. long, supported on concrete piers 20 ft. high. The first set of piers were made in the block yard and placed by the cableways on prepared bases, and braced by steel tie rods. Subsequently larger piers were built in place of mass concrete, 6 ft. square at the base and 4 ft. at the top.

For the first lift there were two parallel traveler tracks which permitted four travelers to be used for each section. These were arranged in pairs, two on each track facing each other on opposite sides of a section, thus eight derricks were available for the section. Two such groups made it possible to work on two sections at the same time. Between the traveler tracks and

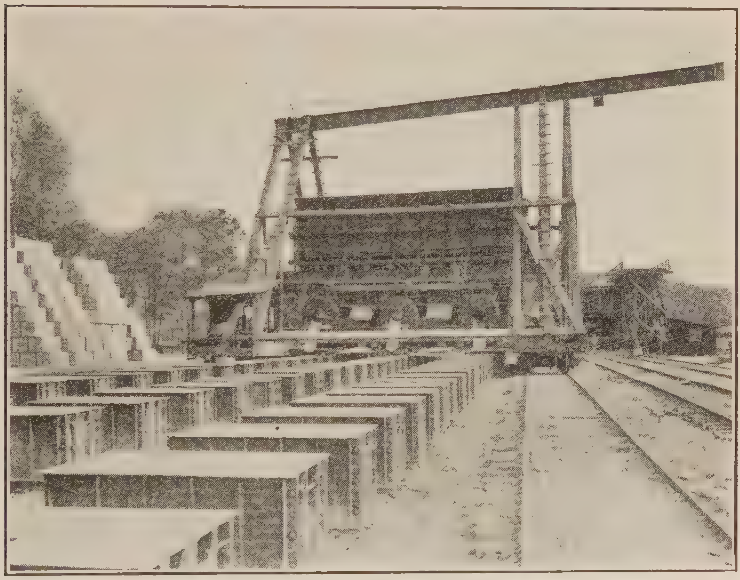

FIG. 8.—TRAVELING MIXING PLATFORM, STEEL FORMS, MATERIAL BINS AND STORAGE PILE OF THE CONCRETE BLOCK PLANT.

at the same elevation, were two service tracks for concrete carried on concrete piers and H-beam stringers. A third service track was placed on the outside of the downstream traveler track, and concrete was served to the upstream derricks over the upstream berm tracks. The track system for the travelers was so arranged that it could be removed from the section in which work was progressing so as not to interfere with the placing of masonry.

The cyclopean stone and facing blocks were delivered by the berm tracks within reach of the outside derricks. The concrete was brought on flat cars from the mixers, located at advantageous points on the hillsides, in bottom-dumping buckets of the Stuebner type of 2 cu. yd. capacity. (Fig. 4.)

These cars, on the elevated tracks, were operated by a hoisting engine and upright boiler carried on one end of the car, the engine being directly connected to one truck axle by chain and sprocket. Those on the berm tracks were usually hauled by a locomotive. Each car usually carried two buckets of concrete from the mixers, and received two empty buckets from the derricks. A mixer of the Hains-Weaver gravity type of 2 cu. yd. capacity was provided for each set of tracks. They were mounted in a timber trestle built across the excavation at the base of the west hill. The trestle was high enough to permit the service tracks running under the chutes. Above the gaging floor were aggregate bins, to which the crushed stone was delivered in 10-cu. yd. side-dumping cars, and the sand in 40-ton gondola cars running on overhead tracks, approached from the downstream side. (Fig. 4.)

During the first season's masonry work, the travelers were used to build sections 6 to 12 on the relatively flat area from the base of the West Hill to the gorge, a length of 550 ft. The preglacial gorge, which occurred on the contact between the gneiss forming the East Hill and the crystalline limestone of the valley bed, was nearly at right angles to the axis of the dam. At the top of the rock sides, the gorge was about 200 ft. wide. It was filled with modified drift to a depth of 80 ft., below which disintegrated rock extended downward between the sound rock sides about 40 ft. more to the solid contact between the two rock formations. The lowest point in the excavation was at El. 63, which was 137 ft. below the floor of the valley, or 307 ft. below the top of the dam.

The portion of the dam to be built in the gorge below the general level of the rest of the foundation contained about 80,000 cu. yd., and it was necessary to place this before the traveler method could be extended for the full length of the dam between the hillsides. For this portion of the dam, the masonry was placed by several 90-ft. guy derricks mounted on each side of the gorge. (Figs. 5 and 6.) A mixing plant on each side of the gorge supplied concrete directly to these derricks. That on the east side was a gravity mixer supplied by an overhead track and trestle from the north. On the west of the gorge, a double-track trestle crossed the foundation at right angles to the axis of the dam at the level of the berm tracks. In the base of this trestle, two 1-cu. yd. electrically driven rotary mixers supplied the west line of derricks. The trestle contained the bins for these mixers, and also served as access to the upstream berm tracks. (Fig. 4.) The cyclopean stone for the gorge was also delivered on this trestle, while short spur tracks on the side of the East Hill served the east line of derricks.

A cut-off trench 20 ft. wide and from 20 to 30 ft. deep, parallel with and near the upstream face, was excavated in the rock below the general level of the foundation the entire length of the dam below El. 300. (Fig. 2.) This trench gave an excellent opportunity for determining the nature of the rock below the general level of the foundation, as the strike of the rock was nearly at right angles to it. The trench was carried down until the bedding seams became tight. Where seams of any importance existed, grout holes were drilled in the rock foundation, usually normal to the dip, which were grouted after a sufficient depth of masonry had been placed. Wherever any seams

FIG. 9.*—GENERAL VIEW OF KENSICO DAM SITE FROM EAST HILL.

* This view, showing the flume, mixing plants for the gorge sections and the upstream face of sections constructed in the gorge, was taken October 30, 1913.

appeared on the surface, they were poured with grout just before placing the masonry. Drainage pipes were put in wherever seepage appeared in the foundation or cut-off trench, and these were subsequently grouted.

The excavation having been carried to satisfactory rock, the foundation was prepared by barring and wedging out all of the loose fragments, and the area cleaned with jets of air and water and by stiff brooming. Just before concrete was placed on any rock foundation, mortar was spread and broomed into all the irregularities of the rock. The concrete in the cut-off trench was 1 : 2 : 4, and the cyclopean stones were restricted to the downstream half of the trench, leaving a 10-ft. width of rich mass concrete on the water side. All of these precautions were to prevent the access of water to the foundation of the dam, although uplift due to water pressure was provided for in the design.

The masonry in the body of the dam was in the proportions of 1 : 3 : 6, and after a depth of 2 ft. had been laid over the rock the cyclopean stones were introduced. These were lowered into beds of soft concrete a foot or so deep and jogged into the concrete by a couple of bars. Large stones weighing more than 6 or 8 tons were supported by the derrick and lowered as the joggling progressed so as to insure perfect bedding.

When the level for beginning the concrete-block facing on the upstream side and the contraction joints was reached, the concrete was leveled off at these places and a course of blocks set. These served as forms for retaining the concrete at these places. The downstream face of the dam was retained by wooden forms. Below the level of the refill, these were sloping in the plane of the downstream face. These forms were made in sections of 2-in. plank, the vertical batten projecting below the bottom of the form and fitted into the horizontal wale at the top of the preceding form. The top of the form was anchored to the masonry at frequent intervals by No. 8 gage steel wire, fastened to steel pins inserted in the previous day's work, or wrapped around projecting cyclopean stone. Above the refill, the downstream face was shaped in steps to receive the stone masonry facing.

As soon as a section had been filled with cyclopean masonry to the top of the block facing, another course of blocks was laid. During the busy part of the season, the blocks were set at night. By this arrangement all the derricks were available during the daytime for cyclopean masonry, and this proceeded uninterruptedly every day.

Concrete Blocks.—The concrete blocks for the upstream face are 2 ft. 5 in. high, 2 ft. 6 in. thick and from 5 to 6½ ft. long. They are arranged in alternate header and stretcher courses. The arrises are finished to a radius of 1 in. They are laid with 1-in. mortar joints. The blocks for facing the contraction joints are 2 ft. 5 in. high and 5 ft. long, with square edges. Their arrangement is shown in the Fig. 7.

About 64,000 cu. yd. of concrete blocks were required, and they were made in a special yard located south of the dam. Two parallel tracks, 43 ft. apart and 1100 ft. long, ran the length of the yard, and carried a 43 x 58-ft. traveling platform supported on two special flat cars. On the platform were mounted three 1-cu. yd. rotary mixers, the necessary material bins and cement house. The bins were supplied with aggregate by a telpher on an

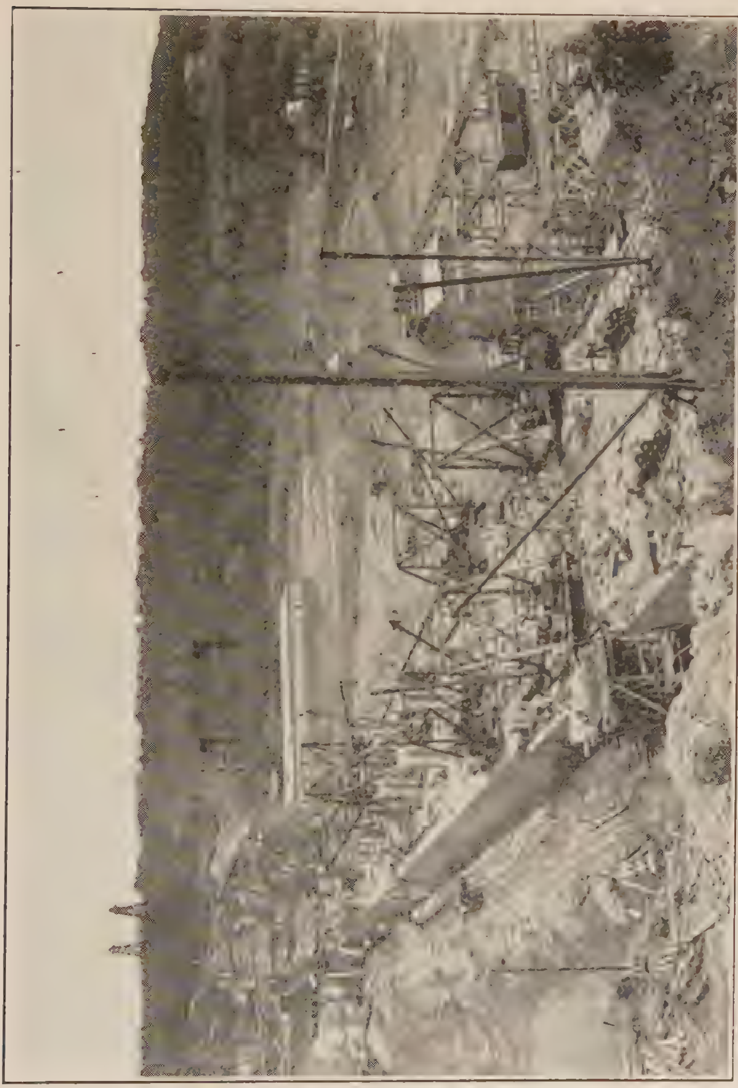

FIG. 10.*—VIEW OF UPSTREAM FACE FROM THE WEST HILL.

* The travelers in the foreground are completing the third lift, those in the background are building the fourth lift. The change from double to single traveler tracks is shown here. View taken May 28, 1914.

overhead cantilever beam, which was supplied with bottom-dumping buckets by train on a parallel track. The buckets were filled at an overhead storage bin on the side of the yard, to which the materials were delivered by trains of side-dumping cars from the crushing plant and sand banks. Between the traveler tracks were three rows of block forms, one row under each mixer on the traveler, so that the mixers discharged the concrete directly into the forms. (Fig. 8.). The concrete was mixed in the proportions of 1 : 3 : 5 to an almost ideal consistency, suggesting well cooked oatmeal. About 9 in. depth of concrete was dumped at a time into the forms and well worked with spading forks. As soon as the three forms were filled, the traveler was moved to the next set of forms by the hoisting engine which operated the telpher. The cement car on a parallel track was moored alongside the traveler and moved with it so that the cement was delivered directly to the mixers.

The forms* were of steel, the sides and ends resting on the bottom plate and all bolted together through the flange angles. The blocks were made face downward, and smooth dense surfaces always resulted. After the concrete had set, the side forms were removed and after the blocks had become sufficiently hard, they were lifted by a locomotive crane, turned over and placed in storage piles to season. The output of this plant averaged 200 blocks, or 240 cu. yd. a day.

Progress.—The first masonry in the dam was placed in the bottom of the gorge on April 23, 1913. In the broad area west of the gorge, the first masonry was placed by the traveler derricks on May 7. At both locations the construction was carried on daily until December 20, when the masonry work was discontinued for the winter.

In the gorge, when the masonry reached the bases of the derricks, they were raised and placed on mass concrete piers at convenient locations, and as the masonry continued upward, the piers were included in the masonry. (Figs. 5 and 6.) Above the bottoms of the contraction joints, these derrick piers were located adjacent to these joints. When the masonry reached the rotary mixers in the trestle on the west side of the gorge, they were removed, located in the trestle at the West Hill and served the traveling derricks. Another gravity unit, located on the East Hill on the upstream side, took their place in supplying concrete for the gorge. By the end of the season, the top of masonry in this vicinity had reached the level of the masonry to the west, so that the following year the traveler method was extended over the whole length between hillsides. The guy derricks were then used to build the closure sections to the hillsides.

In the area between the gorge and the West Hill, a length of 550 ft., the foundation was at about El. 160, and the masonry was placed by the traveler method. A start was made at Section 6 in May with one group of travelers, and in Section 8 in July with the other group. As soon as one section was finished to the base of the traveler tracks, the travelers were moved to the next section. By the first of October the first lift had been completed, the tracks for the second lift erected, the travelers raised to the new track, and

* These forms are described in a paper by A. D. Flinn, Deputy Chief Engineer, printed in *Journal of American Concrete Institute*, June, 1915.

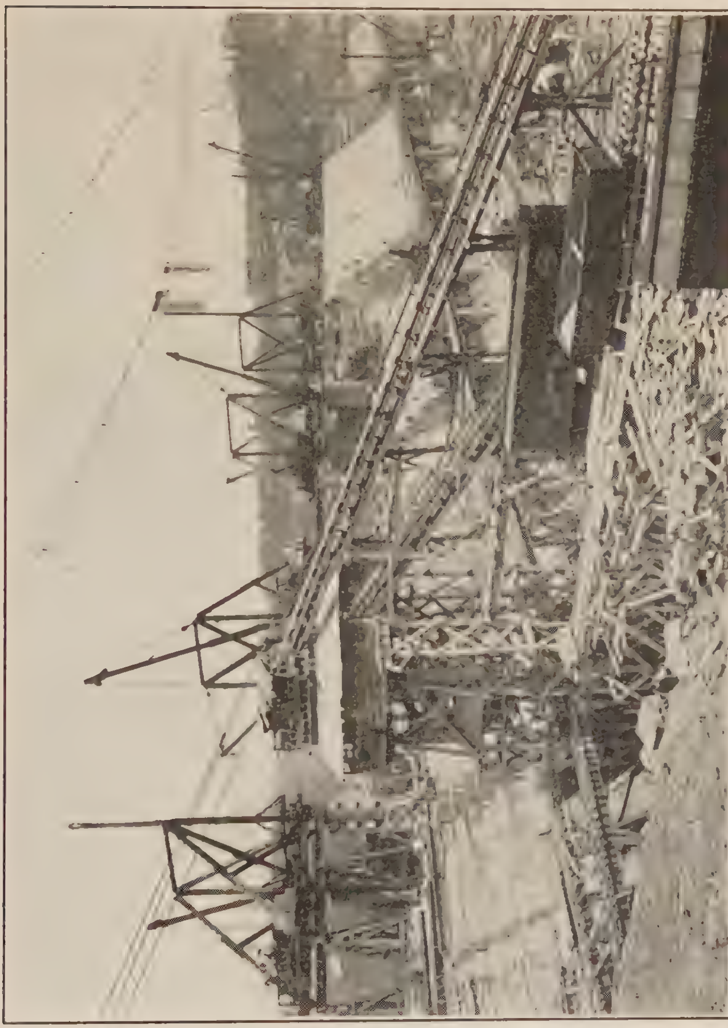

FIG. 11.*—VIEW OF DOWNSTREAM FACE FROM WEST HILL.

* The concrete plant is in the foreground. The service tracks are at the elevation of the terrace, above which the face of the dam is built in steps to receive the facing masonry. View taken July 23, 1914.

work on the second lift was in progress. (Figs. 4 and 9.) The smaller volume in this lift made the progress more rapid, so that by the close of the season it was finished, and the dam completed to a uniform height at El. 214.5 for a

FIG. 12.—CONSTRUCTION OF SECTION 3 AT TOP OF FIFTH LIFT.

[illegible] character of the cyclopean masonry. The travelers in the distance are working on the sixth lift. View taken August 11, 1914.

length of 869 ft. A total of 316,000 cu. yd. of masonry was placed during the season of 200 working days of 8 hours each, or a daily average of 1580 cu. yd. The best day's work was 2250 cu. yd. in place.

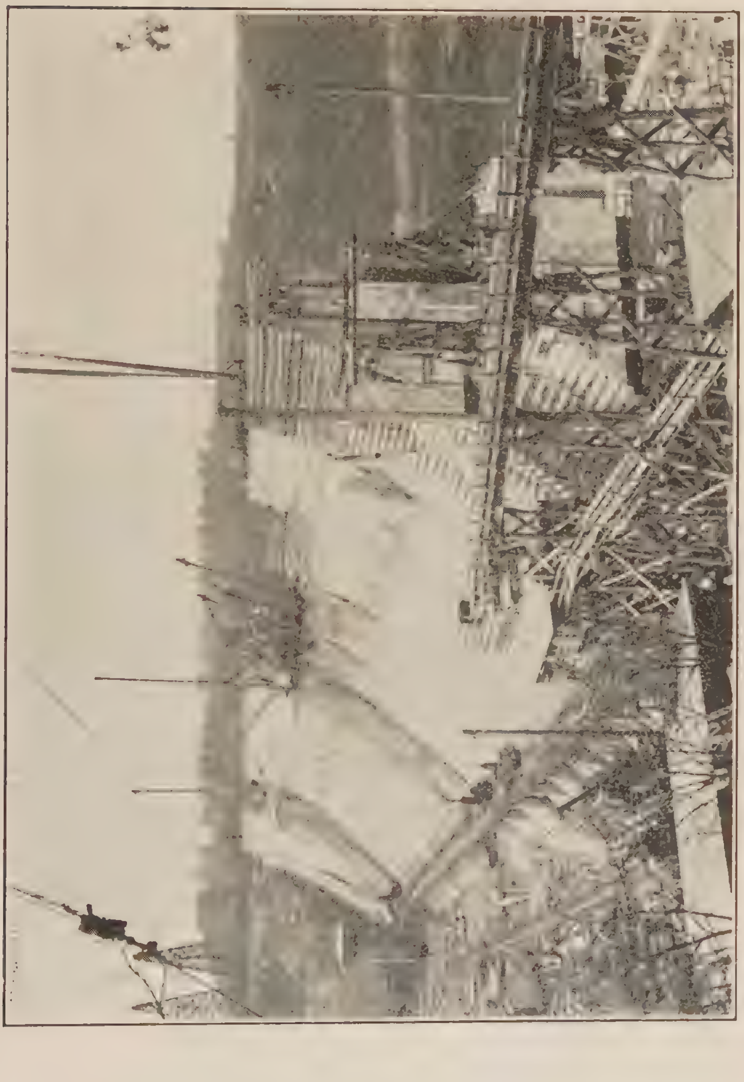

FIG. 13.*—DOWNSTREAM FACE FROM EAST HILL.

* View showing the placing of the top lift by travelers and guy derricks, taken October 29, 1914.

During the winter of 1913–14, the refill on both sides of the dam was placed to the elevation of the completed masonry, so that the entire volume of masonry built during the year was buried. The service tracks were relocated on the fill along both sides of the dam, the traveler tracks for the third lift erected, and the concrete plants relocated to meet the requirements of the coming season. (Fig. 7.) One gravity unit was located at the foot of each hill on the downstream side of the dam, and a third at the East Hill on the upstream side. These were located in timber towers, and the material bins were supplied by belt conveyors drawing from storage bins located at such points that they could be filled by trains running on overhead tracks. The cement came in sacks and was delivered to each mixer directly from the freight cars, either by gravity chutes or belt conveyors. The service tracks ran under the mixers so that the buckets on the shuttle cars were directly under the chutes.

On March 17, 1914, the placing of concrete was resumed. Section after section was rapidly built to the level of the third lift tracks by the traveling derricks, while guy derricks built the closing sections to the hillsides, which enabled the traveler method to be extended for the next two lifts. The third lift was completed the end of May. (Fig. 10.) The top of this lift, El. 239.5, was the same as the terrace and the beginning of the stone facing on the downstream side. At this elevation a bench about 8 ft. wide was left to receive the base of the stone masonry, and above this level the downstream face was formed in steps with risers corresponding to the courses of masonry, about 2.5 ft. at this elevation. (Fig. 11.)

In the fourth lift, El. 264.5, several changes were made. The width of the dam could be covered by the derricks of one traveler, so but one traveler track was erected. This permitted the eight travelers to be grouped in pairs and the construction of four sections at a time. The mass concrete piers for carrying the traveler tracks were replaced by 10-ft. sections of the contraction-joint facing blocks, the gage of the tracks corresponding to a tongue and a groove section of the joint face. The blocks were laid up with their regular bond, the projecting ends of the headers being supported by concrete slabs or filler blocks laid in the same manner as the regular blocks. The intermediate ends of the track girders were supported by timber towers at the center of each section. (Fig. 11.) This change resulted in quite an advantage, as the piers were more easily constructed, involved less cost to the contractor, and the tracks and timber towers could be entirely removed from the sections under construction.

The downstream service tracks were relocated, one on the bench for the facing masonry and the other on the outer line of traveler track piers of the third lift. For the fourth and subsequent lifts, these were used exclusively for concrete from the two downstream mixers, all of the cyclopean stone and blocks being delivered on the upstream side.

The fifth lift, to El. 289.5, was constructed in the same manner as the fourth, except that the booms of the downstream derricks were lengthened 10 ft. in order to reach the outer service tracks. The same arrangement was employed for the sixth and seventh lifts, completing the dam successively

FIG. 14.—PLACING FACE MASONRY ON AUGUST 12, 1915.

to El. 314.5 and El. 339.5. In these lifts the less width of the dam resulted in a greater depth of masonry each day, and two courses of blocks were laid at a time. At the seventh lift, El. 364.5, the piers carrying the downstream traveler rail projected above the face of the dam and were subsequently removed. (Fig. 13.) The eighth and top lift, completing the dam to the top, El. 370, was built by the travelers on the seventh lift track. After one section was built the travelers backed away a section and each was assisted by a guy derrick mounted on the top of the completed section. Finally the last sections occupied by the travelers were built by guy derricks after the travelers were removed.

At the close of the season of 1914, the concrete masonry of the dam had been completed to the top for a length of 1388 ft., 19 completed sections extending from the West Hill to within 300 ft. of the end at the East Hill. A total of 489,750 cu. yd. of masonry had been placed in 221½ working days, an average of 2211 cu. yd. a day for the entire season. The largest day's work was 3572 cu. yd.

During the winter of 1914–15, the excavation for the east end of the dam was finished and preparations made for placing the dimension stone masonry.

During 1915 the dam was completed by finishing the cyclopean masonry east of Joint 19 and placing the facing masonry, the latter amounting to 22,000 cu. yd. of dimension and roughly squared stone. This was placed by five gangs between March 22 and December 9. The traveling derricks were reassembled on the terrace and placed the masonry to El. 280, above which the materials were lifted by these derricks to platforms supported on the face of the dam, and placed by guy derricks mounted on the top of the dam until the frieze course was reached. Above this course, the stones being larger, were lifted by the guy derricks from service tracks on the upstream side of the dam. A rotary mixer mounted on one of the traveler frames furnished the mortar which was discharged into skips carried on a shuttle car on a parallel track. A locomotive ran this car within reach of the traveler derricks. (Fig. 14 and 15.)

Plant.—The placing of over 800,000 cu. yd. of masonry in two working seasons could not have been accomplished without adequate and well-planned plant, operated by an efficient organization and directed by a master mind. Relatively the most important part of the plant was the railroad and its equipment. This was a two-track, standard-gage line from the siding on the Harlem Division of the New York Central Railroad, through the camp and storage yard, crossing the dam site at El. 280 and by two switchbacks north of the dam to the top of the East Hill, where the well equipped shops, repair yard and transformer house were located. From here the railroad continued to the quarry and crusher house about 4000 ft. distant. Connections from the main line led to the service tracks on both sides of the dam, mixing plants and wherever needed. A single-track road about 4 miles long ran from the quarry to the sand pits. Altogether there were about 15 miles of 65-lb. rail, stone-ballasted track. This, and good rolling stock, simplified the problem of delivering to the derricks an average of 4000 tons a day, with a maximum of 7000 tons.

The dam site was covered by two cableways of 1860 ft. span, carried on movable timber towers 125 ft. high. The main cable was $2\frac{1}{2}$ in. diameter. They were erected the first year and assisted in the excavation of the gorge.

FIG. 15.—DETAIL OF FACE MASONRY.

The frieze and cornice stones are anchored to the steel loops projecting from the masonry. View taken September 10, 1915.

Their chief use was in handling and lifting the derrick plant. The concrete in the traveler piers was usually placed by the cableways, but otherwise they were not used for placing masonry. The cableways, derricks, rock drills,

concrete mixers, in fact all of the plant at the dam excepting locomotives, were electrically operated. The power was obtained from the New York Edison Co. over a high-tension power line located along the aqueduct right-of-way from Yonkers.

Quarry.—The stone for the dam was all obtained from a quarry developed about a mile east of the dam on land acquired by the contractor for the purpose. This was so operated as to yield the required amount of cyclopean, dimension and crushed stone practically without waste. A large crushing plant and stone-cutting yard were interesting features of the quarry. The stone is a gneissoid granite of fine grain and interesting color and texture, giving admirable results in both the rough dressing for the face of the dam and the fine-dressed work of the pavilions and other architectural details.

Cyclopean Masonry.—The proportion of large stone in the cyclopean masonry was 27.1 per cent of the whole concrete volume, exclusive of the concrete block facing, the derrick and traveler piers. The cement factor for the same volume was 0.837 bbl. per cu. yd. The cement was, as usual on large work, regularly sampled at the mills and held in the bins pending satisfactory results of tests made in the city's laboratory. The crushed stone was the product of the crusher retained on ½-in. and passing 3-in. diameter circular screen openings. The fine aggregate was bank sand, to which was added from 30 to 50 per cent screenings from the crusher. The actual proportions of the aggregate were based on frequent mechanical analyses for the purpose of obtaining as dense concrete as possible.

One great advantage of the method employed was the distributed concrete mixing units, which resulted in the short lapse of time between the filling of the buckets and their dumping, so that the aggregates did not settle in the buckets and they dumped clean, the concrete being in easily workable condition. The bulk of the concrete was made in the gravity mixers and these were worked to the limit of their capacity at times during the second season. The largest day's work for any one mixer was 653 batches, or 1143 cu. yd. of concrete in place in 8 hours.

As a result of the remarkable progress made in the construction of the dam, the reservoir is available for the full storage of water 3¼ years ahead of the contract time, and the filling began on November 22, 1915, with water from the Ashokan Reservoir in the Catskills flowing through the Catskill Aqueduct.

REPORT OF THE COMMITTEE ON REINFORCED CONCRETE AND BUILDING LAWS.

The Committee on Reinforced Concrete and Building Laws has to report, for the consideration of the Institute, proposed revised Standard Building Regulations for the Use of Reinforced Concrete. These regulations, while following the general form of those adopted in February, 1910, and known at present as the Institute Standard No. 4, are revised and added to in several sections, as the Committee considers present knowledge and experience warrant.

There are two sections of these regulations which one member of the Committee has not been able to subscribe to, and he has made a minority report.

Respectfully submitted by the Committee,

E. J. Moore, *Chairman.*

Minority Report on Standard Building Regulations for the Use of Reinforced Concrete.

As a member of this Committee the writer wishes it clearly understood that he is unalterably opposed to rodded columns as members of a building. He is also opposed to any dependence upon stirrups to take shear or diagonal tension. He is also opposed to any allowance of *compression* in the *tension* coil reinforcing a column.

Exception is therefore taken to all parts of these Regulations that pertain to the above named things.

The writer would recommend that all reinforced columns be round or octagonal and have close-spaced hooping in addition to longitudinal rods; that a standard reinforcement be adopted along these lines and that a flat unit stress be allowed on the full concrete area with no additions for longitudinal steel or hooping.

He would recommend that when reinforcement for diagonal tension or shear is needed it be done by bending up main tension rods and anchoring them fully beyond the edge of support.

In practically all other respects the Report is agreed to, but these features are so big and of such tremendous importance that the writer considers the Regulations a dangerous standard to follow unless the rodded column and the stirrup are eliminated.

Respectfully submitted,

(Signed) Edward Godfrey.

Secretary's Note:—Following the reading of this report, there was a general discussion, after which it was voted to receive the report and refer it back to the Committee for reconsideration and to submit at a later date for formal adoption.

PROPOSED REVISED STANDARD BUILDING REGULATIONS FOR THE USE OF REINFORCED CONCRETE.

I. General.

Definition of Reinforced Concrete. 1. The term "Reinforced Concrete" as used in these Regulations shall mean an approved mixture of Portland cement with water and aggregates in which metal (generally steel) has been imbedded in proportionately small sections, in such a manner that the metal and the concrete assist each other in taking stress.

Use. 2. Reinforced concrete may be used for all classes of buildings if the design is in accordance with good engineering practice and stresses are figured as indicated in these Regulations.

Height of Buildings. 3. There shall be no limit upon the height of buildings of reinforced concrete except as limited by the strength requirements in these Regulations.

Permits. 4. Before permission is granted by the Building Department to erect any reinforced concrete building, complete general plans accompanied by specifications signed by the Engineer or Architect must be filed with the Building Department. Sufficient details shall be included in the plans submitted to make clear the exact dimensions and construction of the reinforced concrete portions of the building and the arrangement of the reinforcement so as to permit computation of all stresses. Specifications shall state the qualities and proportions of the materials to be used.

Copies of approved plans and specifications must be left on file with the Building Department for public inspection until the building is completed.

Inspection. 5. The construction of the building shall be inspected in detail by a representative of the Architect or Engineer who will keep a complete record of the progress of the work, including dates of placing concrete and dates of removing of forms. He shall also check the materials used, and the placing of same in the different parts of the building. These records shall be available for inspection by the Superintendent of Buildings.

Load Tests. 6. The Superintendent of Buildings may require a load test on a floor within a reasonable time after erection. The test shall be made under the supervision of the Superintendent of Buildings and shall show that the construction will sustain safely an applied load of twice the total live load, but in no case less than one and one-half times the total live and dead load.

Posting of Floors. 7. Upon the completion of the building the Architect or Engineer shall, with the approval of the Building Department, issue signed certificates to be posted on each floor of the building stating the safe carrying capacity per square foot.

II. Materials.

Specifications for Cement. 8. Only Portland cement shall be used in reinforced concrete structures. Cement shall meet the requirements of the Standard Specifications for Cement of the American Society for Testing Materials as in effect at the time of the adoption of this Regulation.

9. All cement used shall first be tested by the Architect or Engineer and record of such tests shall be kept at the building site for inspection by the Superintendent of Buildings. No cement which has not met the requirements of the above specifications shall be used without the written approval of the Superintendent of Buildings. **Tests of Cement.**

10. Extreme care shall be exercised in selecting the aggregate for concrete, and careful tests made for the purpose of determining the grading necessary to secure the minimum percentage of voids and the maximum density. **Aggregates—General.**

All aggregates shall be of clean material, free from dust, soft particles, lumps of clay, vegetable loam, and all organic matter.

11. Fine aggregates shall consist of sand, crushed stone or gravel screenings, graded from fine to coarse, and passing when dry a screen having holes $\frac{1}{4}$ in. in diameter, and not more than 6 per cent passing a sieve having 100 meshes per linear inch. **Fine Aggregates.**

12. Mortars composed of 1 part Portland cement and 3 parts of the fine aggregate by weight, when made into briquettes, or into prisms or cylinders, should show a tensile or compressive strength at least equal to the strength of 1 to 3 mortar of the same consistency made with the same cement and standard Ottawa sand. If of lower strength, the proportion of cement shall be increased, but if less than 70 per cent, the fine aggregate shall be rejected. **Test of Fine Aggregates.**

13. Coarse aggregates shall consist of crushed stone, gravel or slag which is retained on a screen having $\frac{1}{4}$-in. diameter holes and graded in size from small to large particles. The maximum size of the coarse aggregate shall be such that the concrete will flow freely around the reinforcement. Bank gravel shall be separated from the sand before mixing. Slag shall be clean, air cooled, blast furnace slag, weighing not less than 75 pounds per cubic foot and containing not more than 1.3 per cent of sulphur as sulphides. **Coarse Aggregate.**

14. Cinders shall not be used as coarse aggregate in concrete for reinforced concrete structures without tests acceptable to the Superintendent of Buildings showing the strength of such concrete. Cinder concrete may be used for fireproofing, for floor and roof slabs, and for partitions. Where cinders are used as the coarse aggregate they shall be composed of hard, clean, vitreous clinker; free from sulphides, unburned coal, or ashes. **Cinders.**

15. The water used in mixing concrete shall be free from oil, acid, alkalies, or organic matter. **Water.**

16. Steel for reinforcement of concrete shall conform to the requirements of the specifications of the American Society for Testing Materials for Concrete Reinforcement Bars, in effect January 1, 1916. **Reinforcement.**

Cold-drawn steel wire made from billets of the grade of rivet steel may be used in floor and roof slabs, column hooping, and for temperature and shrinkage stresses. This steel shall have an elastic limit between 50,000 and 65,000 lb. per sq. in. and an ultimate strength of not less than 85,000 lb. per sq. in.

All reinforcing steel shall be free from excessive rust, scale or coatings of any character which would tend to reduce or destroy the bond.

III. DETAILS OF CONSTRUCTION.

Forms. 17. Forms must be substantial and unyielding and sufficiently tight to prevent the leakage of mortar. Before placing concrete all forms shall be first thoroughly cleaned of all débris and oiled to prevent adhesion of the concrete.

Preparation of Reinforcement. 18. All bars must be carefully bent as required by plans.

Placing of Reinforcement. 19. Reinforcement shall be accurately located in the forms and secured against displacement.

Steel Splices. 20. Where it is necessary to splice reinforcing steel, this shall be done by providing a lap sufficient to transfer the stress between bars by bond and shear, or by a mechanical connection such as a screw coupling. Splices at point of maximum stress should be avoided.

Construction Joints. 21. Vertical fill lines between two fills of concrete must be selected so that the resulting joint will have the least possible effect on the strength of the structure. Before making the second fill the concrete previously placed shall be roughened, thoroughly cleansed of foreign material and laitance, drenched and slushed with a mortar consisting of 1 part Portland cement and not more than 2 parts fine aggregate. Construction joints for columns should be made at underside of floor construction, haunches and column capitals being considered as part of the floor construction. Where reinforced concrete columns have flaring heads, or where structural steel columns are used, concrete for slab and column head may be poured at the same time as the concrete for the column shaft. Intermediate fill lines in columns should be avoided. In general, fill lines in floors should be selected near the center of spans of slabs, beams and girders.

Measuring Ingredients. 22. Methods of measuring of the various ingredients of concrete, including the water, shall be used which will secure separate and uniform measurements of the proportions required. Measurements to be made by volume. 94 lb. of cement to be considered as a cubic foot.

Mixing—General. 23. The ingredients of concrete shall be thoroughly mixed to the desired consistency and the mixing shall continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous.

Machine Mixing. 24. In mixing by machine, a mixer of a type which insures the uniform distribution of the materials throughout the mass shall be used.

Hand Mixing. 25. When it is necessary to mix by hand, the mixing shall be done on a watertight platform, and all ingredients shall be turned together at least six times and until the resulting mass is homogeneous in appearance and color.

Consistency. 26. The materials must be mixed wet enough to produce a concrete of such a consistency that it will flow sluggishly into the forms and about the metal reinforcement, and at the same time can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

Re-Tempering. 27. Mortar or concrete must not be re-mixed with water and used after it has partly set.

Placing of Concrete. 28. Concrete after the completion of the mixing must be transported as rapidly as practicable from the place of mixing to the place of final deposit. The concrete must be deposited in such a manner that it will flow sluggishly around the steel reinforcement and must be rammed or agitated by suitable

tools in such a manner as to produce thoroughly compact concrete of maximum density.

29. Concrete must not be placed in water unless unavoidable; if necessary to do this, excessive mixing-water must be avoided to prevent the cement from being separated from the aggregate. **Placing in Water.**

30. The concrete at the end of each fill shall be cleaned of laitance or other deleterious material which would detract from the quality of the concrete. After forms are removed, any porous sections of concrete shall be cleaned out and filled in a manner to meet the approval of the Superintendent of Buildings. **Finishing.**

31. The face of concrete exposed to rapid drying shall be kept damp for a period of at least five days. **Protection in Warm Weather.**

32. Concrete shall not be mixed or deposited unless it is maintained at a temperature not less than 40° F. during mixing, placing, and for at least 72 hours thereafter, or until the concrete has thoroughly hardened. All materials shall be free from frost before mixing. **Protection in Cold Weather.**

33. Under no consideration shall forms be removed until the concrete has hardened sufficiently to permit their removal with safety. **Removal of Forms.**

34. As soon as a section of form is removed, shoring shall be provided as necessary to carry the weight of the new concrete and other loads brought upon the construction in acting as a support for upper floors. Careful consideration must be given to the loads carried and the strength of the new concrete before any shoring is removed. **Temporary Supports.**

IV. DESIGN.

35. All reinforced concrete construction shall be designed to meet the conditions of loading (including bending in columns) without stressing the materials used beyond the safe working stresses specified. **Conditions.**

36. The dead loads shall be the weight of the permanent structure. The weight of reinforced stone, gravel or slag concrete shall be taken as 144 lb. per cu. ft.; the weight of cinder concrete as 100 lb. per cu. ft. **Dead Loads.**

37. The live load shall be the working or variable load for which the structure is designed. **Live Loads.**

38. All parts of a structure shall be designed to carry safely the entire combined dead and live loads with the exception that the loads on columns and foundations may be reduced by considering that columns in top story carry the total live and dead load above them; columns in next to top story carry the total dead load and 85 per cent of the total live load above; columns in the next lower story, the total dead load and 80 per cent of the total live load above; and thus on downward, reducing at each story the percentage of total live loads carried, by 5, until a reduction of 50 per cent is reached. The columns in this and in every story below this point shall be proportioned to carry the total dead load and at least 50 per cent of the total live load of all the floors and roof above them. **Reduction of Loads.**

39. As a basis for calculations for the strength of reinforced concrete construction the following assumptions shall be made: **General Assumptions.**

(*a*) Calculations shall be made with reference to working stresses and

safe loads rather than with reference to ultimate strength and ultimate loads.

(b) A plane section before bending remains plane after bending.

(c) The modulus of elasticity of concrete in compression within the usual limits of working stresses is constant.

(d) No allowance shall be made for the tensile value of concrete.

(e) Perfect adhesion is assumed between concrete and reinforcement. Under compressive stresses the two materials will, therefore, be stressed in proportion to their moduli of elasticity.

(f) The ratio of the modulus of elasticity of steel to that of concrete shall be taken as 15 for a 1 : 2 : 4 mixture of stone, gravel or slag concrete; 12 for a 1 : 1½ : 3 mixture of same material; and 10 for a 1 : 1 : 2 mixture. For cinder concrete the ratio of the moduli of elasticity shall be taken as 30.

Strength of Materials.

40. Test cylinders of concrete 8 in. in diameter and 16 in. long or 6 in. in diameter and 12 in. long shall develop at the age of 60 days at least the strength in pounds per square inch of section given in the following table:

Aggregate.	Mixture.		
	1 : 1 : 2	1 : 1½ : 3	1 : 2 : 4
Granite, trap rock	3300	2800	2200
Gravel, hard limestone, hard sandstone, and approved slag	3000	2500	2000
Soft limestone and sandstone	2200	1800	1500
Cinders	800	700	600

Safe Working Stresses.

41. Reinforced concrete structures shall be so designed that the stresses, figured in accordance with these regulations, in pounds per square inch, shall not exceed the following:

Extreme fibre stress in concrete in compression—37½ per cent of the minimum compressive strength given in the above table. Adjacent to the support of continuous members, stresses 10 per cent higher may be used, provided the member frames into a mass of concrete projecting at least 50 per cent of the least dimension of the member on all sides of the compression area of the member.

Concrete in direct compression—25 per cent of the minimum compressive strength given in the above table.

Shearing stress in concrete when main steel is not bent and when steel is not provided to resist diagonal tension—2 per cent of the minimum compressive strength given in the above table.

Punching shear in concrete—7½ per cent of the minimum compressive strength given in the above table.

Shearing stress in concrete when steel to assist in resisting diagonal tension is provided—7½ per cent of the minimum compressive strength given in the above table, providing that sufficient web reinforcement is supplied to carry the stresses in excess of the value allowed for the unreinforced concrete; and providing, further, that this web reinforcement extends from top

to bottom of beam and is adequately anchored to the horizontal reinforcement. If main reinforcing bars are bent up and anchored, they may be considered as part of the web reinforcement.

Bond stress between concrete and plain reinforcing bars—4 per cent of the compressive strength.

Bond stress between concrete and approved deformed bars—5 per cent of the compressive strength.

Bearing upon a surface of concrete at least twice the loaded area—50 per cent of the compressive strength of the concrete.

Tensile stress in steel—16,000 lb. per sq. in., except that for steel having an elastic limit of at least 50,000 lb., a working stress of 18,000 lb. per sq. in. will be allowed.

Girder, Beam, and Slab Construction.

42. In determining the bending moment in slabs, beams and girders, the load carried by the member shall include both the dead and the live loads. The span of the member shall be the distance center to center of supports, but not to exceed the clear span plus the depth of the member, except that for continuous or fixed members framing into other reinforced concrete members the clear span may be used. For continuous members supported upon brackets making an angle of not more than 45 deg. with the vertical, and having a width not less than the width of the member supported, the span shall be the clear distance between brackets plus one-half the total depth of the member. If the brackets make a greater angle than 45 deg. with the vertical, only that portion of the bracket within the 45-deg. slope shall be considered.

For members uniformly loaded the bending moment shall be assumed as WL/F, where W = total load; L = span; and $F = 8$ for members simply supported, 10 for both negative and positive bending moment for members restrained at one end and simply supported or partially restrained at the other, and 12 for both negative and positive bending moment for members fixed or continuous at both supports. A special condition of loading to be reduced to equivalent uniformly distributed loading in accordance with approved engineering practice. For members having one end simply supported or partially restrained, at least 50 per cent of the tension reinforcement required at center of span shall be bent up and adequately anchored to take bending moment at exterior support.

Slabs.

43. The main tensile reinforcement shall not be farther apart than $2\frac{1}{2}$ times the thickness of the slab. For slabs designed to span one way, steel having an area of at least $\frac{2}{10}$ of 1 per cent of section of slab shall be provided transverse to main reinforcement, and this transverse reinforcement to be further increased in the top of the slab over girders to prevent cracking, and the main steel in slabs parallel and adjacent to girders reduced accordingly. Where openings are left through slabs, extra reinforcement shall be provided to prevent local cracks developing. This reinforcement shall in no case be less than one-quarter of a square inch in section and must be securely anchored at ends. Floor finish when placed monolithic may be considered part of the structural section.

T Beams and Girders.

44. Where adequate bond and shearing resistance between slab and web of beam is provided, the slab may be considered as an integral part of the beam, but its effective width shall not exceed on either side of the beam $\frac{1}{6}$ of the span length of the beam nor be greater than 6 times the thickness of the slab on either side of the beam, the measurements being taken from edge of web.

Web reinforcement for beams and girders shall be so designed as to adequately take up throughout their length all stresses not taken up by the concrete. Web members shall be spaced not to exceed $\frac{3}{4}$ of the effective depth of the beam in that portion where the web stresses exceed the allowable value of concrete in shear. Web reinforcement, unless rigidly attached, shall be placed at right angles to the axis of the beam or girder, and carried around or securely anchored to longitudinal steel in both the tension and compression areas.

Tile and Joist Floors.

45. Wherever floors are built with a combination of tile or other fillers between reinforced concrete joists, the following rules regarding the dimensions and methods of calculations of construction shall be observed:

(*a*) Wherever a portion of the slab above the fillers is considered as acting as a T-beam section, the slab portion must be cast monolithic with the joist and have a minimum thickness of 2 in.

(*b*) Wherever porous fillers are used which will absorb water from the concrete, care must be taken to thoroughly saturate same before concrete is placed.

(*c*) All regulations given above for beam and girder floors shall apply to tile and joist floors.

(*d*) The sections of fillers shall be together and reasonably tight before concrete is placed.

Flat Slab Floors.

46. Continuous flat slab floors, reinforced with bands or rings of steel and supported on columns having flared heads may be constructed, provided the Architect or Enginner when submitting plans for approval, submits sufficient data obtained from tests on similar floors to justify the proposed design. All floors of this type must be tested in accordance with Section 6 and to meet the approval of the Superintendent of Buildings.

Columns—General.

47. Reinforced concrete columns, for the working stresses hereinafter specified, shall have a width or diameter not less than $\frac{1}{15}$ of the unsupported height nor less than 12 in. All vertical reinforcement shall be secured against lateral displacement by steel ties not less than $\frac{1}{4}$ in. in diameter, placed not farther apart than 15 diameters of the vertical rods nor more than 12 in.

Columns with Longitudinal Reinforcement.

48. For columns having not less than $\frac{1}{2}$ of 1 per cent nor more than 6 per cent of vertical reinforcement, the allowable working unit stress for the gross section of the concrete shall be 25 per cent of the minimum ultimate strength specified in Section 40, and the working unit stress for the steel shall be based upon the ratio of the moduli of elasticity of the concrete and steel.

Columns with Longitudinal and Lateral Reinforcement.

49. Columns, having not less than 1 per cent nor more than 6 per cent of vertical reinforcement and not less than $\frac{1}{2}$ of 1 per cent nor more than 2 per cent of lateral reinforcement in the form of hoops or spirals spaced

not farther apart than $\frac{1}{6}$ of the outside diameter of the hoops or spirals nor more than 3 in., shall have an allowable working unit stress, for the concrete within the outside diameter of the hoops or spirals, equal to 25 per cent of the minimum ultimate value of the concrete, as given in Section 40, and a working unit stress on the vertical reinforcement equal to the working value of the concrete multiplied by the ratio of the specified moduli of elasticity of the steel and concrete, and a working load for the hoops or spirals determined by considering the steel in hoops or spirals as 5 times as effective as longitudinal reinforcing steel of equal volume. The percentage of lateral reinforcement shall be taken as the volume of the hoops or spirals divided by the volume of the inclosed concrete in a unit length of column. The hoops or spirals shall be rigidly secured at each intersection to at least 4 verticals to insure uniform spacing. The percentage of longitudinal reinforcement used shall be not less than the percentage of the lateral reinforcement.

Structural Steel and Concrete Columns.

50. For steel columns filled with concrete and encased in a shell of concrete at least 3 inches thick where the steel is calculated to carry the entire load, the allowable stress per square inch shall be determined by the following formula:

$18{,}000 - 70\,L/R$, but shall not exceed 16,000 lb.—where L = unsupported length in inches and R = least radius of gyration of steel section in inches. The concrete shell shall be reinforced with wire mesh or hoops weighing at least $\frac{2}{10}$ lb. per sq. ft.

Footings—General.

51. Symmetrical, concentric column footings shall be designed for punching shear, diagonal tension and bending moment.

Punching Shear in Footings.

52. The area effective to resist punching shear in column footings shall be considered as the area having a width equal to the perimeter of the column or pier and a depth equal to the depth of footing from top to center of reinforcing steel.

Diagonal Tension in Footings.

53. Shearing stresses as indicative of diagonal tension shall be measured in footings on vertical sections distant from the face of the pier or column equal to the depth of the footing from top to center of reinforcing steel.

Bending Moment in Footings.

54. The bending moment in isolated column footings at a section taken at edge of pier or column shall be determined by multiplying the load on the quarter footing (after deducting the quarter pier or column area) by $\frac{6}{10}$ of the distance from the edge of pier or column to the edge of footing. The effective area of concrete and steel to resist bending moment shall be considered as that within a width extending both sides of pier or column, a distance equal to depth of footing plus $\frac{1}{2}$ the remaining distance to edge of footing, except that reinforcing steel crossing the section other than at right angles, shall be considered to have an effective area determined by multiplying the sectional area by the sine of the angle between the bar and the plane of the section.

Bond Stresses in Footings.

55. In designing footings, careful consideration must be given to the bond stresses which will occur between the reinforcing steel and the concrete.

Walls—General.

56. Walls shall be reinforced by steel rods running horizontally and vertically. Walls having an unsupported height not exceeding fifteen times the thickness may be figured the same as columns. Walls having an unsup-

ported height not more than 25 times the thickness may be figured to carry safely a working load of $12\frac{1}{2}$ per cent of the minimum ultimate load specified in Section 40.

Exterior Walls. 57. Exterior walls shall be designed to withstand wind loads or loads from backfill. The thickness of wall shall in no case be less than 4 in.

Fireproofing. 58. For fireproof buildings, the reinforcement in columns and girders shall be protected by a minimum thickness of 2 in. of concrete; in beams and walls by a minimum of $1\frac{1}{2}$ in.; in floor slabs by a minimum of $\frac{3}{4}$ in.; in footings by a minimum of 3 in.

REINFORCED CONCRETE COLUMNS.

By Pierce P. Furber.*

The report of column tests by the Committee of the American Concrete Institute (see *Journal,* American Concrete Institute, February, 1915, p. 47) is instructive, as it enables one to show by comparison that most of the building ordinances now in effect are far from correct in their specifications for unit stresses on hooped columns. Being published nearly simultaneous with accounts of the Edison fire, this report brought the subject of concrete columns vividly to the attention of those in the building business, regarding questions of both safe loads and fire resistance.

About the most important effect of the Edison fire on the concrete building structures was seen in the supporting columns. In repairing the building, however, it has been found that the columns have proved to be the simplest part of the work. The damage to columns was very serious in some cases, and it has been said that the most urgent lesson to be learned concerns the design of reinforced concrete columns. It would seem to the writer that there is no "lesson" there for the experienced engineer, for the reason that it could have been expected that such columns as those, without hooping of any kind, should have acted very badly in a hot fire.

Supposedly it is well known to structural engineers that columns vertically reinforced and hooped with sufficient spiral reinforcement of close pitch are much to be preferred to columns with vertical rods only, even in case ties are used, as they were not in the Edison buildings. In spite of this knowledge, columns without hooping are being built extensively throughout the country, although it is common practice to use ties or stays of small diameter rods or wire for lateral reinforcement, these being spaced about a foot apart as a rule. The reason for this inferior construction is found in the lower cost of that type as compared to the hooped column, when designed according to most building ordinances.

Many building ordinances place a premium on this dangerous construction by allowing higher relative stresses on the concrete of such columns than on spirally reinforced columns, when we consider the increase in ultimate strength of the latter. Some have gone so far as to permit as great unit pressure on the concrete of a column without hooping as on a well designed hooped column. In other words, the concrete of tied columns is stressed beyond what should be permissible values, and the hooped column, instead of being given the benefit of its increased toughness and reliability, is actually put at a disadvantage because the allowed stresses are lower than tests have shown to be safe.

The results of such building laws are obvious. Designers will use the cheaper type, except in some cases where it may be possible to control the

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design of the work with regard to actual strength secured in the structure rather than with a view to meeting the requirements of some building department. It may be said that a conscientious engineer will always work on the former principle, but this is not so, and will hardly be possible where there is an ordinance limitation. The engineer has no opportunity to use his best judgment. The department officials and framers of the codes have exercised that function so that all one can do is to follow the rules and get the cheapest thing possible.

The discriminating engineer would rather be responsible for the construction of a spiral column—even with unit stresses considerably above those allowed by many building ordinances—than for columns without hooping, under the usual values for unit pressure. The difference between intelligent and inferior design may be judged better by the type of members and details selected than by the figured unit stresses.

Most of the failures of concrete structures have occurred in buildings designed so that the figured unit stresses were not too high when judged by ordinance requirements, and the failures have been due to failures of the type of member rather than of the material. A large number can be blamed on the use of columns without hooping. On the other hand, the writer at least has never heard of the failure of a hooped column in a building. The continued service of hooped columns under pressures, which, according to some building laws, would be considered dangerous, leads one to believe that something is wrong with the building laws, particularly when reliable column tests indicate that the stresses allowed are conservative.

Some may argue that the fact that a building stands up is no sign that it was well designed. This is quite true, and applies with force to many buildings in which rodded columns have been used. In considering designs in the true sense, that is, with reference to the type, it should be the duty of the engineer to avoid members that are subject to sudden failure.

A reproduction of some surves, Fig. 1, presented by Mr. T. L. Condron in a discussion before the Western Society of Engineers,* is offered because reference was made in that discussion to the American Concrete Institute tests which were then in progress. The curves were drawn for the purpose of comparing allowed unit stresses and relative strength of hooped columns according to various building ordinances. The bottom diagram was labeled "Relative Actual Strengths." This was rather misleading, because the percentages represented the figured safe capacities according to ordinances of various cities, and not the actual strengths, which can only be determined by tests. Mr. Condron was reported to have said that the tests being conducted by the Institute Committee would help to establish some satisfactory standard in place of the wide differences then existing between building ordinances.

The writer believes that the tests have proved that most of the building ordinances are too conservative in the unit stresses permitted for hooped columns and a number of them too liberal in the allowances for columns without hooping. Consequently, the cities represented in Mr. Condron's table

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* *Engineering News*, February 19, 1914, p. 435, and *Engineering and Contracting*, February 18, 1914.

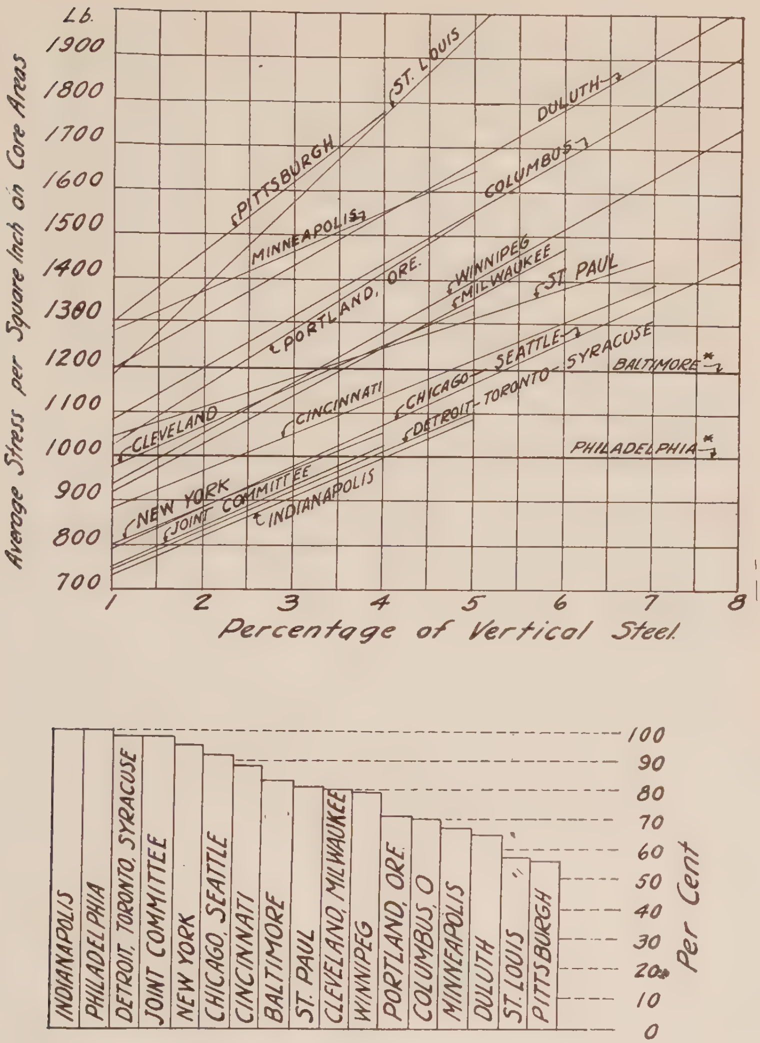

FIG. 1.—COMPARISONS OF ALLOWED UNIT STRESSES (UPPER DIAGRAM) AND OF RELATIVE ACTUAL STRENGTHS (LOWER DIAGRAM) OF 1 : 2 : 4 CONCRETE COLUMNS REINFORCED WITH 1 PER CENT HOOPING AND 4 PER CENT VERTICAL STEEL.

which appear to have radical ordinances are nearer to agreement with the test data than those at the other end of the scale, although the percentages used in computing this table will not produce the most efficient hooped column.

Some building departments have been progressive enough fully to recognize the advantage of hooped columns and have incorporated in their ordinances values in keeping with the strengths disclosed by tests. The tests of the American Concrete Institute have proved the contention that hooped columns are to be preferred and will be cheaper than rodded columns if properly designed. The proof lies in the comparative strength exhibited by the tested columns. It is to be regretted that the tests did not cover a wider range of spiral percentages and also that the concrete mix was $1:1\frac{1}{2}:3$ instead of the standard $1:2:4$ mixture usually employed in building construction.

The writer is submitting in Table I the requirements of a large number of regulations governing the design of concrete columns. The table was made up from the various published building ordinances, some of which do not clearly express the limitations, but they have been interpreted literally as far as possible and the unit stresses are given in most cases so that there can be no misunderstanding.

There are a number of other points of comparison that might be made but which do not affect the unit stresses under discussion to any extent. The requirement for ultimate strength of concrete at certain ages is specified in a large number of the codes. Minimum sizes of columns, ratio of moduli of steel and concrete, maximum spacing and minimum area of vertical rods, minimum number and sizes of rods and unit pressures for columns reinforced with items structural steel are specified by many cities. The requirements for these are various but they are not considered important in the present discussion. There are many different ways of arriving at values for spiral reinforcement and the notes on this table will show some of these methods. A number of the ordinances were undergoing revision at the time this table was made. The writer has taken the latest information he could secure in making up this table.

There are several general methods in vogue for determining the proportions of columns.

Rules based on the report of the Joint Committee are used quite generally. Rochester, Newark, Toronto and Louisville have ordinances very near these requirements.

The Chicago ordinance has been copied largely by other cities, notably Seattle, Omaha and Winnipeg, and in a modified form by Detroit.

A third general class consists of those cities whose codes are based on Considere's formula and researches. Minneapolis, St. Paul and Columbus are examples of this class of cities.

Another method is that of Philadelphia in specifying a fixed value for the concrete when a minimum percentage (1 per cent) of spiral is used and nothing is allowed on vertical steel. The value for hooped concrete is double that allowed on columns without hooping.

A number of other cities, as San Francisco, Milwaukee, Indianapolis and Cleveland, specify unit stresses for concrete with a minimum percentage of

spiral and allow no value for the spiral directly. The vertical steel is allowed a stress of nf_c, where n is the ratio of moduli of steel and concrete. This is somewhat along the line of the Joint Committee recommendations but variations in the stresses are usually made. Cincinnati allows no greater value on the concrete of hooped columns than on tied columns, though the stress allowed on hooping is supposed to take care of the increase in strength.

In comparing the various requirements, it is remarkable to find such general disagreement among the framers of ordinances. There is much room for improvement and a need for revision of a large part of the present laws. This is especially true when we consider that it should be well known that many of the ordinance values do not correspond with values determined from tests of columns.

A study of the ultimate strengths obtained in the American Concrete Institute tests should certainly be profitable to those who have advocated such values for columns as were recommended by the Joint Committee. One could ask no better evidence of the value of hooping or the economy to be obtained by the use of hooped columns than the data submitted. If columns were to be designed on the basis of their actual efficiency under load, there would be few engineers attempting to use the rodded column for anything but minor posts or columns carrying light loads.

The Chicago ordinance on columns has been appropriated by many because it was in many respects a thorough specification. This has been particularly true of cities in the Middle West, which have followed quite generally anything that has been done in Chicago. The values are conservative and the general theory of that ordinance agrees with the opinions of a great many. The unit stresses used, however, will not check out accurately as reasonable working values if we consider them in the light of published test data. It would be well to call attention to a "joker" in the Chicago ordinance which might readily be overlooked, and, in fact, has been overlooked by many until pointed out specifically. Interpretation of the ordinance is based on the assumption that the modulus of elasticity of concrete is considered unchanged by the addition of hooping, and as the "action of the hooping may be assumed to increase the resistance of the concrete equivalent to $2\frac{1}{2}$ times the amount of the spiral hooping considered as vertical reinforcement," it is seen that the stress on vertical steel is increased.* This is expressed in the formula:

$$f'_s = nf_c\ (1+2.5\ np')$$

as the value of vertical steel in compression, where f_c is the allowable concrete stress, $n = E_s/E_c$ and p' is the ratio of hooping. This means that the unit stress allowed on the steel, instead of being nf_c, as a reading of the ordinance would imply, is greater by the amount $2.5\ np'$, which allows a value for 1:2:4 concrete and $1\frac{1}{2}$ per cent hooping, of

$$15\times500\ [1+(2.5\times15\times0.015)] = 11{,}720.$$

The wording of the ordinance would indicate a value of 15×500, or 7500 lb. Similarly for $1\frac{1}{2}$ per cent hooping with 1 : $1\frac{1}{2}$: 3 concrete, the value would be

* *Engineering News*, June 6, 1912, p. 1070.

TABLE I.—REQUIREMENTS OF DIFFERENT BUILDING REGULATIONS AND AUTHORITIES REGARDING REINFORCED CONCRETE COLUMNS.

(The notes referred to in this table are printed on page 188.)

L=length of column; D=least diameter of column; d=least diameter of core; d'=least diameter of vertical rods; n=ratio of modulus of elasticity of steel to that of concrete.

Authority.	Mix.	Unit Compression, Plain Concrete, lb. per sq. in.	Columns with Vertical Reinforcement and Ties. f_c Unit Concrete Compression, lb. per sq. in.	Verticals. f_s Unit Compression, lb. per sq. in.	Verticals. p Minimum Per Cent.	Verticals. p Maximum Per Cent.	Ties. Maximum Spacing, in.	Ties. Minimum Size of Rod, in.	Hooped Columns. f_c Unit Concrete Compression, lb. per sq. in.	Hooping. f_h Unit Value as Imaginary Verticals in Compression, lb. per sq. in.	Hooping. p' Minimum Per Cent.	Hooping. p' Maximum Per Cent.	Hooping. S. Maximum Pitch, in.	Hooping. Minimum Wire, in.	Verticals. f_s Unit Compression, lb. per sq. in.	Verticals. p Minimum Per Cent.	Verticals. p Maximum Per Cent.	Maximum Permissible Unit Compression on Core, lb. per sq. in.	Minimum Fireproofing Outside Steel, in.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1. C. A. P. Turner, 1914	1:2:4	..	350	10,000	..	..	$10d'$[1]	¼ rd.	800	2.4×16,000	0.5	..	3	¼ rd.	12,000	..	8.0	2,000	1½
	1:1½:3	..	..	..	..	..	..	..	960	..	..	..	..	..	10,000	..	8.0	..	..
2. Joint Committee, 1910	1:6	[2]	652[3]	9,780	1.0	4.0	$0.25D$	..	652	0	1.0	..	$0.25D$	..	9,780	1.0	4.0	..	2
3. Am. Conc. Inst., 1910	1:6	650	450	6,750	..	..	$0.25D$	..	540	0	1.0	..	$0.25D$	..	6,750	..	..	..	[4]
	..	..	..	..	..	..	..	..	650	0	1.0	..	..	..	9,750	1.0	4.0	..	2
4. Nat. Bd. Fire Und., 1915	1:6	500	500[3]	6,000	0.5	3.0	$15d'$[5]	¼ rd.	750	0	1.0	..	$\frac{1}{6}d$[6]	¼ rd.	9,000	1.0	4.0	..	2
	1:4½	600	600	7,200	..	..	..	..	900	..	..	..	..	..	10,800	..	..	..	..
5. Boston, 1912[7]	1:6	416	416	4,160	..	..	..	..	..	..	..	..	..	..	..	..	..	..	1½
	1:5	370	370	3,700	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
6. Providence, 1913[8]	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
7. New Haven, 1914	1:2½:5	..	500[3]	6,000	0.5	1.0	$\frac{1}{4}D$	1%	1,000	0	1.0	..	$\frac{1}{8}d$	[9]	12,000	0.5	1.0	..	1½
8. Waterbury, 1914	1:2:4	400	400[3]	0	0.5	..	12	¼ rd.	600	[10]	..	..	$\frac{1}{7}d$	¼ rd.	0	0.5	..	..	2
9. New York, 1915	1:6	500	500	7,500	0.5	4.0	[5]	..	500	[49]	0.5	2.0	[6]	..	7,500	1.0	4.0	..	2 [11,12]
10. Rochester, 1914	1:6	650	450	6,750	..	..	$\frac{1}{4}D$[13]	..	650	0	..	..	$0.25D$	..	9,750	1.0	4.0	..	1½ [4]
	..	..	650	9,750	1.0	4.0	..	..	..	..	..	..	..	..	..	..	..	..	..
11. Syracuse, 1913	1:2:4	..	500[3]	7,500	1.0	..	D, 12	..	650	0	1.0	..	..	..	9,750	..	..	..	.. [4]
12. Buffalo, 1915[14]	1:2:5	350	350	4,200	..	..	D	..	..	..	..	..	..	..	..	..	..	..	..
13. Newark, 1911	1:2:4	..	450	6,750	..	..	D, 12	¼ rd.	540	0	1.0	..	..	¼ rd.	8,100	..	..	..	2
	..	..	..	..	..	..	..	..	650	0	1.0	..	..	..	9,750	1.0	4.0	..	..
14. Philadelphia, 1914[15]	..	500	500	0	..	..	d	..	1,000	0	..	..	$\frac{1}{9}d$[16]	..	0	..	..	1,000	2 [17]
15. Baltimore, 1908	1:2:4	400	500[3]	0	..	6.0	12	..	1,200	0	..	..	..	..	0	..	..	1,200	2 [18]
16. Pittsburgh, 1913	..	450	540	8,100	1.0	4.0	[19]	⅛ rd.	[20]	[20]	0.5	..	[6]	⅛ rd.	..	1.0	4.0	..	1½
17. Cleveland, 1911[15]	1:2:4	500	500	10,000	1.0	10.0	12	..	500	..	..	..	..	..	10,000	1.0	10.0	..	2
18. Toledo, 1912[21]	1:2:4	350	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
19. Detroit, 1914	1:1½:3	400	550	6,600	0.5	4.0	$12d'$	..	650	18,720	0.5	1.5	$0.1d$[22]	..	7,800	0.5	8.0	..	1½
	1:1:2	..	..	..	..	..	..	..	750	21,600	..	..	..	..	9,000	..	..	..	..
20. Columbus, 1913	1:2:4	..	500	7,000	0.5	4.0	[23]	0.625 sq. in.	700	2.4×10,000	0.5	1.5	[24]	$\frac{5}{16}$ rd.	10,000	..	..	..	2 [25,26,27]
	..	..	..	..	..	..	..	..	..	2.4×12,000[28]	..	..	..	..	..	..	..	..	..
21. Cincinnati, 1913	1:2:4	200	600[3]	10,800	1.0	..	12	0.0625 sq. in.	600	2.2×10,800	1.0	..	1¾	0.0625 sq. in.	10,800	1.0	..	..	1
	1:1½:3	..	700[3]	10,500	..	..	..	..	700	2.2×10,500	..	..	..	..	10,500	..	..	..	..
22. Louisville, 1913	..	650	650	9,750	1.0	4.0	12	..	650	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	9,750	1.0	4.0	..	[illegible]

Authority.	Mix.	Unit Compression, Plain Concrete, lb. per sq. in.	Columns with Vertical Reinforcement and Ties. f_c Unit Concrete Compression, lb. per sq. in.	Verticals. f_s Unit Compression, lb. per sq. in.	Verticals. p Minimum Per Cent.	Verticals. p Maximum Per Cent.	Ties. Maximum Spacing, in.	Ties. Minimum Size of Rod, in.	Hooped Columns. f_c Unit Concrete Compression, lb. per sq. in.	Hooping. f_h Unit Value as Imaginary Verticals in Compression, lb. per sq. in.	Hooping. p' Minimum Per Cent.	Hooping. p' Maximum Per Cent.	Hooping. S Maximum Pitch, in.	Hooping. Minimum Wire, in.	Verticals. f_s Unit Compression, lb. per sq. in.	Verticals. p Minimum Per Cent.	Verticals. p Maximum Per Cent.	Maximum Permissible Unit Compression on Core, lb. per sq. in.	Minimum Fireproofing Outside Steel, in.
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
	1:2:4	440	450	6,750	0.5	4.0	12d', 12	..	640	0	1.0	1.5	⅙d, 2½	..	9,600	p'	5.0	..	2
23. Indianapolis, 1913	1:1½:3	..	540	6,480	..	..	..	..	768	..	..	..	..	..	9,216	..	..	..	..
	1:1:2	..	652	6,520	..	..	..	..	928	..	..	..	..	..	9,280	..	..	..	1½ [26 27 32]
24. Grand Rapids, 1913	1:2:4	[29]	600[3]	8,000	[30]	[30]	[31]	0.1104 sq. in.	800	2.4×16,000	..	..	⅙d	0.0767 sq. in.	10,000	..	..	..	1½
	1:2:4	..	400	6,000	0.5	3.0	12d', 18	..	500	18,750	0.5	1.5	0.1d, 3	..	[33]	p'	8.0	..	..
25. Chicago, 1911	1:1½:3	..	480	5,760	..	..	..	..	600	18,000	..	..	..	..	[33]	..	..	..	..
	1:1:2	..	580	5,800	..	..	..	..	725	18,125	..	..	..	..	[33]	..	..	..	..
	1:2:4	[34]	500[3]	7,500	..	..	12	⅜ rd.	800	0	1.0	..	3	¼ rd.	12,000	1.0	6.0	..	1½
26. Milwaukee, 1914	1:1½:3	..	..	..	..	..	..	..	900	..	..	..	..	..	13,500	..	..	..	..
	1:2:4	..	500[3]	7,500	1.0	..	20d'	0.03 sq. in.	500	2.4×14,000	..	..	0.1d	0.03 sq. in.	14,000	.	.	..	2 [35]
27. St. Louis, 1914	..	..	..	..	..	..	..	..	..	2.4×20,000	..	..	..	..	20,000	.	..	..	.. [28]
28. Kansas City, 1915[21]	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
29. Minneapolis, 1911	1:2:4	[29]	600[3]	8,000	[30]	[30]	[31]	0.1104 sq. in.	800	2.4×16,000	0.5	1.5	⅙d	0.0767 sq. in.	10,000	.	5.0	1,830	1½ [25 26 27 34]
30. St. Paul, 1910	1:4	[29]	500	7,500	..	..	12	¼ rd.	750	2.4×16,000	0.5	2.0	[24]	¼ rd.	10,000	p'	7.0	..	2 [32]
	1:2:4	350	500[3]	8,000	[37]	[37]	[23]	0.1104 sq. in.	600	2.4×16,000	..	..	⅛d, 7d'	0.0767 sq. in.	12,000	..	..	..	1½ [27 38 39 40]
31. Duluth, 1912	1:2:4	..	600[3]	8,000	[30]	[30]	[31]	..	..	..	..	..	..	..	..	..	..	..	..
	1:1½:3	..	..	..	..	..	..	..	750	..	..	..	..	..	..	..	..	..	..
	1:2:4	..	400	6,000	0.5	3.0	12d', 18	..	500	18,750	0.5	1.5	0.1d, 3	..	[33]	p'	8.0	..	1½
32. Omaha, 1912	1:1½:3	..	480	5,760	..	..	..	..	600	18,000	..	..	..	..	[33]	..	..	..	..
	1:1:2	..	580	5,800	..	..	..	..	725	18,125	..	..	..	..	[33]	..	..	..	..
33. Denver, 1905[21]	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
	1:2:4	..	450[3]	6,750	0.5	5.0	[41]	¼ rd.	500	18,750	0.5	1.6	[24]	¼ rd.	[33]	p'	7.5	..	1½ [42]
34. Seattle, 1914	1:1½:3	..	540	6,480	..	..	..	..	600	18,000	..	..	..	..	[33]	..	..	..	..
	1:1:2	..	652	6,520	..	..	..	..	725	18,125	..	..	..	..	[33]	..	..	..	..
35. Portland, Ore., 1911	1:2:4	500	500	7,500	1.0	4.0	D	..	750	2.4×11,250	0.5	1.5	⅙d	..	11,250	p'	5.0	..	2
36. San Francisco, 1910	1:6	500	500	7,500	1.0	5.0	d	0.05 sq. in.	700	0	..	..	0.1d	..	10,500	1.0	5.0	..	2[43]
	1:2½:3½	[44]	550	8,250	1.0	5.0	d, 15d'	3/16	800	0	0.5	..	3	[45] 3/16	12,000	1.0	7.5	..	1½
37. Los Angeles, 1914	1:3:4½	[44]	440	6,600	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
	1:2:4	650	650	9,750	0.5	..	D, 12	¼ rd.	800	[45]	1.0	..	⅐d	¼ rd.	12,000	1.0	..	1,100	1½ [45]
38. Memphis, 1913	1:1½:3	750	750	9,000	..	..	..	..	900	..	..	..	..	..	10,800	..	..	1,300	..
	1:1:2	850	850	8,500	..	..	..	..	1,070	..	..	..	..	..	10,700	..	..	1,500	..
39. Richmond, 1912	1:2:4	500	500	7,500	1.0	10.0	12	..	..	..	..	..	..	..	..	..	..	..	2 [46]
40. Atlanta, 1911	1:2:4	..	600[47]	6,000	..	..	D	5/16 rd.	..	..	..	..	..	..	..	..	..	..	1 [48]
41. New Orleans, 1913[21]	..	500	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
42. Toronto, 1913[15]	1:2:4	..	450[3]	6,750	1.0	4.0	d, 12	¼ rd.	650	0	1.0	..	⅙d, 3	¼ rd.	9,750	1.0	4.0	..	2 [4]
43. Winnipeg, 1913	1:2:4	..	500	7,500	0.5	3.0	12d', 18	..	600	22,500	0.5	1.5	0.1d, 3	..	[33]	p'	8.0	..	2

NOTES TO TABLE I.

[1] $10d'$ or 9 in.

[2] 450 for $\left(\frac{l}{D}<12\right)$.

[3] On core.

[4] If a richer mix is used, increase the concrete unit stress in proportion to the increase in ultimate strength, but not more than 25 per cent.

[5] $15d'$ or 12 in.

[6] $\frac{1}{6}d$ or 3 in.

[7] Rules regarding columns may be formulated by the Commissioner of Buildings.

[8] No requirements for unit stresses.

[9] Diameter in inches $=\frac{1}{8}S$.

[10] For cylindrical columns reinforced with bands or spirally wound hooping, allow on the hooping 3.14 times lateral resistance of hoops when stressed to not more than 16,000 lb. per sq. in., provided there is enough hooping to insure a lateral resistance of 65 lb. sq. in., but not to exceed 100 lb. per sq. in.

[11] Increase concrete stress 20 per cent for 1:1½:3 mix.

[12] Wire for hooping must be drawn from billets and have an ultimate strength of at least 85,000 lb. per sq. in.

[13] $\frac{1}{4}D$ or 12 in.

[14] No specifications for hooped columns.

[15] Ordinances being revised when this table was prepared.

[16] $\frac{1}{8}d$ or 3 in.

[17] Spiral and vertical steel must be sufficient to develop a factor of safety of 4 on concrete. The area of hooping per inch of height must be $W/65{,}000\pi d$, where W is the total load.

[18] The percentage of hooping and vertical steel (p and p') must be sufficient to develop a factor of safety of 6.

[19] $16d$ or the distance center to center of corner rods.

[20] The Pittsburgh code has the following formula for the strength of hooped columns:

$$\left(A_c+nA_s\right)\ \left(750+4.8\frac{16{,}000A_y}{Sh}\right),$$

where A_c is the area of the core in square inches, A_s is the area of the vertical steel, A_y the area of the hooping steel, S is the pitch and h is the diameter of the core in inches. For exterior columns the units are reduced 20 per cent.

[21] No specifications.

[22] $0.1d$ or 3 in.

[23] 12 in. or half the distance center to center of verticals.

[24] $\frac{1}{7}d$ or 3 in.

[25] For exterior columns, reduce units 20 per cent.

[26] Properly welded bands may be used in place of spiral; spacing of bands not to exceed $7d'$.

[27] Spirals must be continuous, machine made and provided with accurate spacers in order to use these values.

[28] High carbon steel.

[29] With L/D less than 5,500; with L/D greater than 5 and less than 12,300.

[30] Eight rods.

[31] $7d'$ or 12 in.; two sets.

[32] If wire not less than $\frac{5}{16}$ in. diameter is used for spiral, allow 25,000 instead of 16,000 in Column 11.

[33] Varies with p', viz: $nf_c(1+2.5np')$.

[34] 400 for L/D less than 8.

[35] Medium steel.

[36] p and p' may be greater than tabulated, the allowed unit stresses to be determined by the Building Inspector.

[37] Four rods.

[38] If the spiral wire is at least $\frac{5}{16}$ in., the ultimate strength is 80,000 lb. per sq. in. and the elastic limit is 50,000 lb. per sq. in., allow 20,000 instead of 16,000 in Column 11.

[39] If S is more than $0.2d$, reduce the unit stresses 50 per cent of the proportional increase of spacing.

[40] Concrete unit stresses may be increased 15 per cent for concrete 2 years old.

[41] $15d'$, d or 18 in.

[42] Spirals must have three spacers.

[43] Hooping must be designed to resist tension due to unit lateral pressure of one-fifteenth of the unit compression on concrete.

[44] Not allowed for columns.

[45] Hooped columns to be computed according to rules approved by Building Commissioner.

[46] For buildings over four stories high, steel must carry 75 per cent of dead load at 1000 lb. per sq. in., except where in buildings of any height concrete will carry $LL+DL$ with f_c less than 500. Steel must not be less than 1 per cent or more than 10 per cent.

[47] This value is for columns carrying balanced loads; for columns with balanced loads on two sides and one unbalanced load, the value is 500, and for columns with two unbalanced loads it is 400.

[48] p must be at least 1.5 for columns where L/D is 12 to 14, at least 1.25 for $L/D=10$ to 12, and at least 1.0 where L/D is less than 10.

[49] Hooping stress not greater than two times unit tension allowed on steel, not greater than 20,000 lb. per sq. in., and not greater than one-third the elastic limit for drawn wire.

10,440 instead of 7200, and for 1:1:2 concrete with $1\frac{1}{2}$ per cent hooping, the value would be 9970, instead of 7250. The interpretation of the ordinance thus changes the values by quite a wide margin. It is questionable as to how far interpretation may be made by department officials, and the writer would raise the point that the ordinance should be interpreted as it reads, for otherwise the officials of the department are exercising a discretion not given them by the statute.

The Considere formula based on many experiments by that eminent French engineer, and others, considers the three elements, concrete, vertical steel and hooping, as contributing to the strength of a column. From his experiments he deduced values for these three elements. He recommended the following formula:

$$P=1.5\ A_cC+pA_cF_s+2.4qA_cF'_s$$

in which P represents the ultimate strength; A_c, the area of column core; C, the ultimate compressive strength of plain concrete; F_s, the elastic limit of the vertical steel; F'_s, elastic limit of spiral; p, the ratio of vertical steel to area of core, and q the corresponding ratio of spiral reinforcement. The value of the concrete is taken as 1.5 times the value for plain concrete, because the tests showed an excess strength which Considere could not account for, so he raised the allowance on the concrete by 50 per cent. The ratio of 2.4 as the proportionate resistances of spiral and vertical steel was determined from his tests on pipes filled with sand. This ratio has been quite generally accepted as a fair value.

Compared with the Considere formula, the formula recommended by the American Concrete Institute Committee is significant. Quoting from page 76 of the *Journal of the American Concrete Institute* for February, 1915, the formula reads as follows:

$$f=3000\ (1-p)+40{,}000p+160{,}000\ (1-10p)q.$$

In this f is the average ultimate unit stress over the whole column section, p is the ratio of vertical steel and q is the ratio of spiral. (The reason for the factor $(1-10p)$ is not clear.)

If in Considere's formula we assume 1:2:4 concrete with a strength of 2000 lb. per sq. in., the elastic limit of vertical steel as 40,000 and the elastic limit of spiral as 60,000 for the wire usually used in spirals, we have

$$P=3000A_c+40{,}000pA_c+144{,}000qA_c,$$

or the ultimate unit bearing is

$$f=3000\ (1-p)+40{,}000p+144{,}000q.$$

This is nearly the same as the formula recommended by the Institute Committee, although it should be noted that the Institute formula was deduced for 1:$1\frac{1}{2}$:3 concrete.

The working stresses permitted by the Minneapolis ordinance, for example, may be expressed in a similar formula.

$$f=1000\ (1-p)+10{,}000p+38{,}400q.$$

This is for 1: 1½: 3 concrete. The first term would be 800 $(1-p)$ for 1:2:4 concrete. It appears that the factor of safety with the Minneapolis values is easily between 3 and 4, which is conservative.

The values used and advocated by Mr. C. A. P. Turner, of Minneapolis, in his book,* and given in Table I, correspond with the Minneapolis ordinance in the case of the load being applied through the concrete, by splicing of vertical rods, except for the value of 12,000 lb. per sq. in. allowed for vertical steel. It should be noted that the units are reduced for columns with metallic end bearings. In the tests of the American Concrete Institute the loads were applied through metal-bearing plates. If Turner's contention be correct regarding the effect of such bearings, the ultimate strength of the test columns would have been somewhat greater under the condition obtaining in a practical building.

Applying the above formula to the case of a 20-in. core, 1 per cent spiral, 2 per cent vertical, we have

CONSIDERE.

20-in. core	314–6 = 308 sq. in. at 3000	924,000
1 per cent spiral	3.14 sq. in. at 144,000	452,000
2 per cent vertical	6.28 sq. in. at 40,000	251,000
		1,627,000

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$3.14\,(1-(10\times.02))\times 160{,}000$	924,000
	402,000
	251,000
	1,577,000

The ultimate strengths given for columns Nos. 3, 3A and 3B, which had 1 per cent of spiral and 2 per cent of vertical steel, were 1,737,500, 1,647,000 and 2,025,000 lb., respectively, giving an average unit compressive strength of 5245 lb. over the total area of column including concrete outside of spiral. For the same column the Minneapolis ordinance allows working compression as follows:

	1:2:4	1:1½:3
20-in. core	246,000	308,000
1 per cent spiral	120,000	120,000
2 per cent vertical	63,000	63,000
Totals	429,000	491,000

This indicates a factor of safety of about 3½, which is as great as is required for safe construction. When we consider that the safety factor ordinarily secured in structural steel designing is actually about 2 to 2½ on the elastic limit, the factor of 3½ seems conservative on hooped columns.

As evidence of the safety of the Minneapolis ordinance, there has never been any concrete failure there as far as the writer knows, and it may be added that rodded columns are a rarity. These two facts are significant and the coincidence is not peculiar. Other cities where the same conditions obtain have as good a record.

Comparing hooped columns and rodded columns, we find that the Chicago ordinance allows an increase of 25 per cent for the bearing value of the concrete of hooped columns over the values allowed for columns with vertical steel only. A factor of safety of 5 is assumed for the latter. Thus,

* "Concrete Steel Construction," Eddy and Turner, 1914, p. 318.

for 1 : 2 : 4 concrete assumed to have a compressive strength of 2000 lb. per sq. in., the unit stress allowed is 400 lb. per sq. in. For a hooped column with $\frac{1}{2}$ of 1 per cent of spiral, the allowance would be, not counting vertical steel,

$$500+(2.5\times 15\times 500\times .005)=593 \text{ lb. per sq. in.}$$

With $1\frac{1}{2}$ per cent hooping, the allowable unit stress would be

$$500+(2.5\times 15\times 500\times .15)=781 \text{ lb. per sq. in.}$$

Assuming 2 per cent vertical steel to be used for the purpose of comparison, the above values would be multiplied by the factor, $1-p'+(n-1)p$,* or 1.275 for $p'=0.005$ and 1.265 for $p'=0.015$, giving 755 and 987 as the pressures respectively for $\frac{1}{2}$ and $1\frac{1}{2}$ per cent hooping. Assume now an ultimate strength of 5000 lb. per sq. in. for the former and 6000 lb. for the latter. The factors of safety would be 6.6, and 6.1, respectively. The factor is increased about 30 per cent over that permitted for unhooped concrete, in spite of the fact that the hooped column is tougher and more reliable, yields gradually and could safely be allowed considerable reduction in the margin of safety.

The article mentioned above* was a discussion of "Concrete Column Economics." All columns treated in this paper were theoretically designed and compared as to cost. The author, J. Norman Jensen, states that "the most economical hooped column is always more expensive than the most economical reinforced concrete column," also that "the cheapest reinforcement is cement." (By "reinforced concrete column" is meant one with vertical rods and no hooping.)

These conclusions emphasize the remarks made herein relative to selection of types. All of the conclusions are based on figures compiled in accordance with the Chicago ordinance. While it appears from the text of Mr. Jensen's article that the first statement above quoted is correct, examination of the figures shows that columns compared for the same load, if made of the same size, would be cheaper with spiral reinforcement. But the assumption as to costs and the proportions of the columns were not such as to enable one to form a conclusive opinion as to the relative properties of the two types, except for Chicago conditions. As Mr. Jensen has pointed out, however, they indicated the trend in Chicago at that time, which was to use columns without spiral except in cases of heavy loads where the size was limited. I believe it is still the general practice to use columns without hooping if they are cheaper.

The observation that cement is the cheapest reinforcement has been made a great many times, and such a statement might lead some to advocate the use of columns without any steel reinforcement. The difference in safety between plain concrete and columns with only vertical reinforcement would be not much, if any, greater than the difference between rodded and hooped columns, but the plain column would be enormous in size as compared to the hooped column. In order to design columns without hoop-

* *Engineering News*, June 6, 1912, p. 1070.

ing so that they would come within the size given for hooped columns, a large increase in vertical steel would be required. In practice the size would undoubtedly be increased a few inches in order to eliminate the cost of the vertical steel as well as the cost of the spiral, but the saving would be made at the expense of safety both during construction and afterward, as well as

TABLE II.—COMPARISON OF ALLOWABLE COLUMN LOADS.

Column: 24 in. outside diameter; 1 : 2 : 4 mix, except as noted; bearing values given in 1000-lb. units.

Authority.	Mix.	Core Diameter, in.	Bearing Value on Net Area.	Verticals.		Spirals.			Total Bearing Value.
				Number.	Bearing Value.	Wire, in.	Pitch, in.	Bearing Value.	
Turner	..	21	270	8–$1\frac{1}{8}$ in. rd.	95	$\frac{3}{8}$	2	139	504
" Metal Bearings	..	"	236	"	80	"	"	139	455
Joint Committee	..	20	199	"	78	"	"	0	277
N. A. C. U.	..	"	199	"	78	"	"	0	277
Nat. Bd. Fire Und.	..	"	229	"	71	"	"	0	300
New Haven	1:$2\frac{1}{2}$:5	21	338	"	95	"	"	0	433
Waterbury, Conn.	..	20	184	"	0	"	$1\frac{7}{8}$	60	244
New York	..	"	153	"	60	"	"	137	320
Rochester	..	21	220	"	78	"	2	0	298
Syracuse	..	"	220	"	78	"	"	0	298
Newark	..	20	199	"	78	"	"	0	277
Philadelphia	..	"	314	"	0	"	"	0	314
Baltimore	..	"	415	"	0	"	"	0	415
Pittsburgh	..	21	(formula)	"	..	"	"	..	437
Cleveland	..	20	153	"	80	"	"	0	233
Detroit	1:$1\frac{1}{2}$:3	21	220	"	62	"	"	67	349
Columbus—									
Medium steel	..	20	214	"	80	"	$1\frac{7}{8}$	86	380
High carbon steel	..	"	214	"	80	"	"	104	398
Cincinnati	..	21	203	"	86	"	2	85	374
Louisville	..	"	220	"	78	"	"	0	298
Indianapolis	..	20	196	"	76	"	"	0	272
Grand Rapids	..	21	270	"	80	"	"	139	489
Chicago	..	"	169	"	83	"	"	67	319
Milwaukee	..	"	270	"	95	"	"	0	365
St. Louis—									
Medium steel	..	20	153	"	111	"	$1\frac{7}{8}$	121	385
High carbon steel	..	"	153	"	159	"	"	173	485
Minneapolis	..	21	270	"	80	"	2	139	489
St. Paul	..	20	229	"	80	"	$1\frac{7}{8}$	140	449
Duluth	..	21	203	"	95	"	2	139	437
Omaha	..	"	169	"	83	"	"	67	319
Seattle	..	"	169	"	83	"	"	67	319
Portland, Ore.	..	20	229	"	89	"	$1\frac{7}{8}$	97	415
San Francisco	..	"	214	"	83	"	2	0	297
Los Angeles	..	21	270	"	95	"	"	0	365
Memphis	..	"	270	"	95	"	"	104	*381
Richmond	..	20	153	"	59	"	"	0	212
Atlanta	..	21	203	"	47	"	"	0	250
Toronto	..	20	199	"	78	"	"	0	277
Winnipeg	..	"	184	"	101	"	$1\frac{7}{8}$	82	367

* Maximum.

in the case of fire. The column would "pass" the building department and be pronounced a first-class design if the unit stresses prescribed were not exceeded.

It is a usual requirement of building ordinances that the spiral be limited to $1\frac{1}{2}$ per cent of the section, but vertical steel is allowed to run as high as 8 or 10 per cent. A higher percentage of spiral and less vertical would be

more scientific. A percentage of spiral equal to about half the percentage of vertical steel will be a proper proportion, and columns so designed will show the greatest strength. The spiral should be not less than $\frac{1}{2}$ to $\frac{3}{4}$ of 1 per cent in any case.

The writer has made up a series of tables, Nos. II to VI, to show the inconsistencies of our column ordinances. The basis of Table II was a column of 24 in. outside diameter, designed according to the Turner specification to carry a load of approximately 250 tons. Vertical steel amounts to about 2.3 per cent and spiral 1 per cent. The core is figured as 21 in., allowing $1\frac{1}{2}$ in. of fireproofing. Some ordinances require 2 in. of fireproofing, in which case the pitch is decreased in case a value is allowed for the spiral, giving the same weight of spiral. Where no value is allowed for the spiral, but a minimum of 1 per cent is required, the pitch is not changed. In this

TABLE III.—COLUMNS OF RICHER MIXTURES THAN 1:2:4.

Compared for Same Size Columns and Steel as in Table II. Bearing Values in Thousands of Pounds.

Authority.	Mix.	Core Diameter, in.	Bearing Value on Net Area.	Verticals.		Spirals.			Total Bearing Value.
				Number.	Bearing Value.	Wire, in.	Pitch, in.	Bearing Value.	
Turner	1:1½:3	21	324	8–1⅛ in. rd.	95	⅜	2	139	558
" Metal Bearings	..	"	284	"	79	"	"	139	502
Nat. Bd. Fire Und.	1:4½	20	283	"	86	"	"	..	369
Detroit*	1:1½:3	21	220	"	62	"	"	67	349
"	1:1:2	"	253	"	71	"	"	78	402
Cincinnati	1:1½:3	"	236	"	83	"	"	83	402
Indianapolis	1:1½:3	20	235	"	73	"	"	..	308
"	1:1:2	"	284	"	74	"	"	..	358
Chicago†	1:1½:3	21	203	"	75	"	"	65	343
"	1:1:2	"	245	"	63	"	"	65	373
Milwaukee	1:1½:3	"	304	"	107	"	"	..	411
Duluth	1:1½:3	"	253	"	95	"	"	139	487
Memphis	1:1½:3	"	304	"	86	"	"	93	450‡
"	1:1:2	"	362	"	85	"	"	92	519‡

* Values for 1:1½:3 mix from Table II.
† Omaha and Seattle figures are same as those of Chicago.
‡ Maximum.

case the weight is a trifle less, due to the smaller core. The Joint Committee design is an example. This table affords a direct comparison of the loads allowed by the various ordinances, the last item in the table being the comparison desired. The average bearing on a 21-in. core for a load of 500,000 lb. would be 1443 lb. per sq. in., and with a 20-in. core it would be 1590 lb. per sq. in. Assuming an ultimate strength of 5500 lb. per sq. in., we would have a factor of safety of nearly 4 for a column with a 21-in. core.

Table III is a continuation of the comparison, using richer mixtures, as specified by some ordinances. It should be noted that the Detroit ordinance does not provide for 1:2:4 concrete. Hence, the values for Detroit in Table II are for a 1:$1\frac{1}{2}$:3 mixture. The city of New Haven specifies 1:$2\frac{1}{2}$:5 concrete with no unit stresses for richer mixtures.

Table IV is a comparison of columns using the same size as for Table II, the steel being increased to bring the capacity of the columns up to the capacity of the Turner column. This table includes different mixtures, as specified.

TABLE IV.—PROPORTIONS OF COLUMNS TO CARRY SAME LOAD AS TURNER COLUMNS, IF POSSIBLE, WITHOUT CHANGING SIZE OF COLUMN.

Mix 1 : 2 : 4 Unless Noted, Bearing Values in Thousands of Pounds.

Authority.	Mix.	Core Diameter, in.	Bearing Value on Net Area.	Verticals.		Spirals.			Total Bearing Value.
				Number.	Bearing Value.	Wire, in.	Pitch, in.	Bearing Value.	
Turner	..	21	270	8-1⅛ in. rd.	95	⅜	2	139	504
Joint Committee	..	20	196	12- "	116	"	" 1%	0	312
N. A. C. U.	..	"	196	12- "	116	"	" "	0	312
Nat. Bd. Fire Und.	1:6	"	226	12- "	108	"	"	0	334
"	1:4½	"	272	12- "	129	"	"	0	401
New Haven	..	21	332	14- "	168	"	" 1%	0	500
Waterbury	..	20	203	4-¾ in. rd.	0	"	1¾	69	272
New York	..	"	151	12-1⅛ in. rd.	90	½	2 2%	245	486
Rochester	..	21	216	14- "	136	⅜	"	0	352
Syracuse	..	"	205	31- "	300	"	"	0	505
Newark	..	20	196	12- "	116	"	"	0	312
Philadelphia	..	"	314	6-¾ in. rd.	0	"	" 1%	0	314
Baltimore	..	"	415	8-1⅛ in. rd.	0	"	"	0	415
Pittsburgh	..	21	†	12- "	..	"	1¾	..	506
Cleveland	..	20	141	32- "	318	"	2	0	459
Detroit	1:1½:3	21	209	25- "	194	7/16	1⅞ 1½%	97	500
"	1:1:2	"	248	16- "	143	"	" "	112	503
Columbus—									
Medium steel	..	20	206	19- "	189	"	2 1½%	112	507
High carbon steel	..	"	209	16- "	159	"	" "	135	503
Cincinnati	1:2:4	21	197	18- "	188	"	"	117	502
"	1:1½:3	"	232	15- "	156	"	"	113	501
Louisville	..	"	216	14- "	136	⅜	"	0	352
Indianapolis	1:2:4	20	190	16- "	152	"	" 1%	0	342
"	1:1½:3	"	229	16- "	145	"	"	0	374
"	1:1:2	"	276	16- "	145	"	"	0	421
Grand Rapids	..	21	270	8- "	80	"	1¾	158	508
Chicago*	1:2:4	"	162	22- "	247	7/16	1⅞ 1½%	99	508
"	1:1½:3	"	194	22- "	218	"	"	95	507
"	1:1:2	"	238	22- "	173	"	"	95	506
Milwaukee	1:2:4	"	261	20- "	239	⅜	2 1%	0	500
"	1:1½:3	"	298	15- "	202	"	"	0	500
St. Louis—									
Medium steel	..	20	153	10-1⅛ in. rd.	139‡	7/16	1½	210	501
High carbon steel	..	"	153	8- "	159‡	⅜	1¾	189	501
Minneapolis	..	21	270	8- "	80	"	"	158	508
St. Paul	..	20	229	8- "	80	7/16	1⅞	192	501
Duluth	1:2:4	21	203	8- "	95	"	"	202	500
"	1:1½:3	"	253	8- "	95	⅜	1¾	158	506
Portland, Ore.	..	20	225	14- "	157	7/16	2 1½%	126	508
San Francisco	..	"	209	16- "	167	⅜	" 1%	0	376
Los Angeles	..	21	261	20- "	239	"	" "	0	500
Memphis	1:2:4	"	..	8-¾ in. 1%	..	"	" "	..	381
"	1:1½:3	"	..	"	..	"	" "	..	450
"	1:1:2	"	..	"	..	"	" "	..	519
Richmond	..	20	141	32-1⅛ in. rd.	238	"	"	0	379
Atlanta	..	21	174	55- "	328	"	"	0	502
Toronto	..	20	196	12- "	116	"	"	0	312
Winnipeg	..	"	176	20- "	242	"	1⅞	82	500

* Omaha and Seattle figures same as those of Chicago.
† Given by formula.
‡ Medium steel is figured at 14,000 lb. and high carbon steel at 20,000 lb.

In the case of some of the ordinances, the percentage allowed limits the total pressure to values less than are necessary to carry 250 tons. In such cases a 24-in. column cannot be figured to carry the required load. Attention is

called to the case of Atlanta requiring fifty-five 1⅛-in. rods inside of a 21-in. core. This is approximately 17 per cent reinforcement. Computations for Atlanta were made on the basis of balanced loads. There is no limitation as to percentage of steel and no limitation as to spacing of vertical rods, except that they must be uniformly spaced about the circumference of the column or at the corners in the case of square or rectangular sections, and about 2 in. from the outer face. Since the circumference of core would be 66 in. and the bars, if arranged on the perimeter, would occupy 62 in., the situation is an interesting one. It is a fact that very few of the ordinances referred to in this discussion make any specification whatever as to the position of the vertical steel, although it is usually understood that the steel should be placed in a

TABLE V.—PROPORTIONS OF COLUMNS TO CARRY 500,000-LB. LOAD.
Mix 1 : 2 : 4 Unless Noted; Bearing Values in Thousands of Pounds.

Authority.	Mix.	Size.		Bearing Value.	Verticals.		Spirals.			Total Bearing Value.
		Total, in.	Core, in.		Number.	Bearing Value.	Wire, in.	Pitch, in.	Bearing Value.	
Turner..........	1:2:4	24	21	270	8-1⅛ in. rd.	95	⅜	2	139	504
Joint Committee.	..	29	25	307	20- “	195	1% “	“	0	502
N. A. C. U.......	..	29	25	307	20- “	195	“ “	“	0	502
Nat. Bd. Fire Und.	1:6	29	25	354	17- “	153	“ “	“	0	507
“	1:4½	27	23	361	13- “	140	“ “	“	0	501
Waterbury.......	..	33	29	396	*8-¾ in.	0	7/16	“	113	509
New York.......	..	25	21	167	12-1⅛ in. rd.	90	2% ½	“	256	513
Rochester........	..	28	25	307	20- “	195	1% ⅜	“	0	502
Newark..........	..	29	25	307	“	195	“ “	“	0	502
Philadelphia.....	..	30	26	530	8-¾ in. rd.	0	“ “	“	0	530
Baltimore.......	..	27	23	500	8-1⅛ in. rd.	0	“ “	“	0	500
Cleveland........	..	25	21	155	35- “	348	“ “	“	0	503
Louisville........	..	28	25	307	20- “	195	“ “	“	0	502
Indianapolis.....	1:2:4	29	25	301	21- “	200	“ “	“	0	501
“	1:1½:3	28	24	333	19- “	174	“ “	“	0	507
“	1:1:2	26	22	335	18- “	166	“ “	“	0	501
San Francisco....	..	28	24	303	19- “	198	“ “	“	0	501
Memphis........	1:2:4	28	25	..	†8-1 in. rd.	..	“ “	“	..	540
“	1:1½:3	26	23	..	†8-⅞ “	..	“ “	“	..	539
Richmond.......	..	27	23	206	40-1⅛ in. rd.	298	“ “	“	0	504
Toronto.........	..	29	25	307	20- “	195	“ “	“	0	502

* 0.5 per cent. † 1 per cent.

circle just inside of the spiral. The writer has seen columns in Chicago with two sets of bars placed in concentric circles, the outer set being just within the spiral.

In the case of Atlanta no spiral reinforcement is required by the Atlanta ordinance, which specifies that there shall be hoops or bands spaced not further apart than the width of the column. The spiral is shown in Tables II and III merely to correspond with the requirements for the typical column which is used as a basis.

Table V gives the sizes of columns for cities whose ordinances would not allow the 24-in. column of Table IV to carry a load of 250 tons. The size is increased in this table to a minimum which will pass for the required load.

Table VI is based on a 24-in. square column with four rods, one in each corner. Vertical steel is tied together with small rods, as noted. Some ordinances allow gross areas to be figured. In most of the ordinances it is not

clear whether gross or net areas are allowed, so those which are not definite are figured as net areas. It will be noted that the values for Minneapolis and Grand Rapids are based on the requirements for columns with eight vertical

TABLE VI.—BEARING ALLOWED FOR A 24-IN. SQUARE RODDED COLUMN WITHOUT SPIRALS.

All Values are for Net Area inside Ties, Unless Noted; Vertical Steel, Four 1⅛-In. Rounds, 3.96 Sq. In., 0.9 Per Cent; 1 : 2 : 4 Concrete unless Noted; Bearing Values in Thousands of Pounds.

Authority.	Mix.	Core, in.	Bearing Values.		Ties.		Total Bearing Value.
			Concrete.	Vertical Steel.	Size, in.	Spacing, in.	
Turner	..	21	153	39	¼	9	192
Joint Committee	..	"	285	39	"	6	324
N. A. C. U.	..	20	178	27	"	"	205
Nat. Bd. Fire Und.	1:6	"	198	24	"	12	222
"	1:4½	"	238	29	"	"	267
Boston	..	21	238†	16	..	..	254
New Haven	..	"	218	24	⅜	2¼	242
Waterbury, Conn.	..	20	158	0	¼	12	158
New York	..	"	286†	24	"	"	310
Rochester	..	21	284	39	"	6	323
Syracuse	..	"	218	30	"	12	248
Buffalo	..	21	200†	17	"	24	217
Newark	..	20	178	27	"	12	205
Philadelphia	..	"	198	0	"	21	198
Baltimore	..	21	218	0	"	12	218
Pittsburgh	..	"	236	32	⅛	18	268
Cleveland	..	20	198	39	¼	12	237
Detroit	1:1½:3	21	315†	26	"	"	341
Columbus	..	20	198	28	5/16	12	226
Cincinnati	1:2:4	22	289	43	"	"	332
"	1:1½:3	"	337	42	"	"	379
Louisville	..	21	284	39	¼	"	323
Indianapolis	1:2:4	20	178	27	"	"	205
"	1:1½:3	"	214	26	"	"	240
"	1:1:2	"	258	26	"	"	284
Grand Rapids	..	21	262(?)	32	⅜	"	294(?)
Chicago*	1:2:4	"	175	24	¼	"	199
"	1:1½:3	"	210	23	"	"	233
"	1:1:2	"	253	23	"	"	276
Milwaukee	..	"	218	30	⅜	"	248
St. Louis	..	20	198	30	¼	22	228
Minneapolis	..	21	262(?)	32	⅜	12	294(?)
St. Paul	..	20	198	30	¼	"	228
Duluth	..	21	218	32	⅜	"	250
Portland, Ore.	..	20	198	30	¼	24	228
San Francisco	..	"	198	30	5/16	21	228
Los Angeles	..	21	240	33	¼	16	273
Memphis	1:2:4	"	284	39	"	12	323
"	1:1½:3	"	328	36	"	"	364
"	1:1:2	"	371	34	"	"	405
Richmond	..	20	198	30	"	"	228
Atlanta	..	21	262	24	5/16	24	286
Toronto	..	20	178	27	¼	12	205
Winnipeg	..	"	198	30	"	"	228

* Omaha and Chicago figures same as those of Chicago.
† Figured on gross areas; see text of paper.

rods. There are no specifications for the type of column compared in this table.

In explanation of the tables, the writer would call attention to the fact that a number of cities, such as New Orleans, Toledo, Providence, Denver,

Kansas City, had no specifications at the time this table was made up. It should be observed that the city of Boston has no ordinance covering this subject, but their rules may be formulated by the Building Commissioner. The writer questions the legality of such procedure.

A peculiar method is employed in Buffalo. There are no ordinances governing reinforced columns, or in fact reinforced concrete in general, but the plans for each building are made the subject of a special ordinance. It is necessary to pass this ordinance through the city council in order to secure a permit for the building.

The writer was unable to secure any information regarding the ordinances of Washington, Worcester, Birmingham, Dayton, Nashville and Jersey City. He was advised that the ordinances of Montreal and Albany were being revised and that none of the old regulations were available. The specifications of "The National Association of Cement Users" were taken from *Engineering Record*, March 5, 1910, page 268. It seems to the writer that these regulations which are still quoted on the authority of the American Concrete Institute should be revised and brought up to date in order to correspond more closely with the tests published by the Committee of this Institute.

Applying the Institute formula to the Turner column of Table III, we find

24-in. column, 21-in. core, 338 sq. in.	1,014,000
Verticals, 8–1⅛-in. rd., 7.95 sq. in.	318,000
Spiral ⅜-in. rd. 2-in. p., 3.61 sq. in.	444,000
Total	1,776,000

Value from the table is 502,000, which shows a factor of safety of 3.55. The Chicago column for the same proportion shows a bearing value of 343,000 giving a factor of safety of 5.2.

The formula of the American Concrete Institute for columns without spiral hooping, as in Table VI, is

$$f = 3000\,(1-p) + 40{,}000p.$$

(See *Journal*, American Concrete Institute, February 15, 1915, p. 69.) On this basis the column of Table VI would figure as follows:

24-in. column, 21-in. core, 437 sq. in. net at 3000	1,311,000
4–1⅛-in. rd., 3.96 sq. in. at 40,000	158,000
Total	1,469,000

The Turner column is given a bearing of 192,000, or factor of safety of 7.6, the basis being 1:2:4 concrete, however. Chicago column bearing value, 1:1½:3 concrete, 233,000; factor of safety of 6.3. The factor for columns without hooping is increased over that for hooped columns more than double by the Turner specification, and about 20 per cent by the Chicago ordinance. The Turner allowance for a column without hooping is much

less than most of the ordinances. Only one (Waterbury) is less than Turner's allowance, and that ordinance permits no load to be figured on the vertical steel. On the other hand the permissible load for hooped columns by the Turner formula is greater than the allowances of any ordinance in the list. The differentiation between types is thus very marked, the margin of safety for the less reliable member being increased largely, as correct principles demand.

Compare the series of columns marked 2, 2a and 2b with nominally 1 per cent each of spiral and vertical. The section is approximately 20 in. in diameter, vertical steel 4–1-in. rounds, spiral $\frac{3}{8}$-in. rounds, $2\frac{1}{4}$-in. pitch with four spacers, concrete $1:1\frac{1}{2}:3$ mixture, average ultimate strength 4920 lb. per sq. in. By the Minneapolis ordinance the safe strength would be figured at 460,000 lb., or 1465 lb. per sq. in., giving a factor of 3.4. By the Chicago ordinance the allowances would be

20-in. core, $314-3.12=311$ sq. in. at 600.............	186,600
$\frac{3}{8}$-in. rd. spiral $2\frac{1}{4}$-in. p., 3.06 sq. in. at 18,000.........	55,080
Total..	241,680
$(241680/314)\times 12=9240$ lb. per sq. in., bearing allowed on vertical steel four 1-in. rounds at 9240.........	28,830
Total..	270,510

or 861 lb. per sq. in., giving a safety factor of 5.7.

In any comparison between the Chicago ordinance values and those of Minneapolis, or most any other building ordinance for columns, it should be noted that the allowable percentage of reduction for live load is greater according to the Chicago ordinance. The Schneider specification of 5 per cent reduction of live load per floor except for the top floor and roof is an accepted standard in most cities. In Chicago the live load for the top floor is reduced 15 per cent and each floor below that 5 per cent additional. For heavy buildings this amounts to a considerable portion of the total load. Consequently, the Chicago ordinance values in practical work are applied to smaller column loads than would be used in other cities.

The series of 5 and 5a of the American Concrete Institute tests contain the only columns in which more than 1 per cent of spiral was used. The values were higher than any others in the table. Comparing No. 2 and No. 5 averages, it is seen that the 1 per cent increase in spiral adds 27 per cent to the ultimate strength. An increase of 1 per cent vertical in column tests No. 3 adds only 6.6 per cent to the average ultimate unit pressure recorded for No. 2. It should be observed, also, that the figures for ultimate strength are based on the total section of column, including a thin sheet of concrete outside the spiral.

It is unfortunate that the high spiral percentage was not tested out in company with higher vertical percentages. We would expect a considerable increase in strength with the vertical percentage made about twice that of

the spiral, or at least equal to it. One fortunate thing about the tests was that the verticals consisted of large rods, although for the higher percentages the rods were larger than would ordinarily be used. Usually $1\frac{1}{4}$-in. round rods are as large as should be used to secure the best results.

Tests made at Phœnixville for Mr. C. A. P. Turner are instructive in that the percentages of steel used were such as to give very high strength, although the columns are not strictly comparable to the American Concrete Institute tests, because the hooping consisted of welded bars and light spirals instead of the relatively heavy spirals used in the American Concrete Institute tests. It is thought that the effect would be equivalent, however. The Phœnixville tests are fully described in "Concrete Steel Construction," by Eddy and Turner, page 316. Ultimate strengths from 5800 lb. per sq. in. on columns with $\frac{3}{4}$ per cent spiral and 4 per cent vertical to 8850 lb. per sq. in. on columns with $2\frac{1}{2}$ per cent spiral and 8 per cent vertical were developed. With 2 per cent spiral and 7 per cent vertical, the ultimate strength was 7600 lb. per sq. in. These tests did not cover a very wide range. Concrete was mixed in proportions of 1 part cement, 1 part sand, $1\frac{1}{2}$ parts buckwheat gravel and $3\frac{1}{2}$ parts gravel ranging from $\frac{1}{4}$ to $\frac{3}{4}$ in. This is practically a 1 : 2 : 4 mix. The columns were built by a contractor on the building site and could be said to represent practical conditions.

It is believed that the above comparisons and computations, in conjunction with the report on the tests of the American Concrete Institute committee and many other tests of columns will show conclusively that columns designed in accordance with the Turner formula will give conservative values for hooped columns. It would be well for many of our building departments to bring their column ordinances up to date in the specifications for hooped columns and thereby to discourage the use of the column without hooping, which has already proved itself a source of danger in too many disasters which might well have been avoided if properly hooped columns had been used.

The specification of reasonable values for reinforced concrete hooped columns will tend to the increase of fireproof construction and a corresponding reduction of inflammable timber construction, because the difference in cost between scientifically designed concrete construction and well designed timber mill construction will be small in the ordinary warehouse or factory building. On a big job concrete frequently will be cheaper. Many times, the difference in cost is prohibitive under present conditions. The writer does not advocate using high stresses merely to reduce cost, but he does believe that it is time for building departments and framers of ordinances to recognize the true strength of hooped columns and to provide laws which will permit reasonable stresses in such members.

TESTS ON CONCRETE COLUMNS, PLAIN AND REINFORCED.

By Frank P. McKibben* and A. S. Merrill.†

This paper records the results of tests on 33 concrete columns made in the Fritz Engineering Laboratory by the civil engineering department of Lehigh University.

The columns were made and tested by the following students: Messrs. A. K. Hohl, W. H. Mohr, Henry Reimers, L. C. Wright, A. E. Hunt, J. A. Sosnowski and G. R. Wood. Mr. S. H. Ingberg, who at the time was instructor in civil engineering at Lehigh University, had charge of this work in the laboratory, and to him and the students is due the credit for work well done.

In view of recent important improvements made in concrete columns at Lehigh University it seems desirable to publish this paper at the present time; notwithstanding the fact that the 33 columns were tested some time ago.

The cement was donated by the Lehigh Portland Cement Company.

Outline of Tests.

Table I is a tabular view of the specimens, showing number and size of the different columns, their reinforcement and results of tests.

The columns, about 14 in. in diameter and from 5 to 20 ft. in length, were of sizes used in buildings. The 1 : 2 : 4 concrete, although not sufficiently rich to secure greatest strength, was nevertheless of a common mixture.

Of the 33 columns, 9 were plain and 24 were reinforced. The group of 9 plain concrete columns included 3 which were approximately 5 ft. long, 3 of 10-ft. lengths, and 3 of 20-ft. lengths. Of the 24 reinforced specimens 3 were 5 ft. long, 18 of 10-ft. lengths and 3 of 20-ft. lengths. The 24 reinforced columns were arranged in eight sets of three each, with:

One set having 0.46 per cent spiral reinforcement.

Three sets having 0.96 per cent spiral reinforcement, but with 5-, 10- and 20-ft. lengths, respectively.

One set having 1.95 per cent spiral reinforcement.

One set having 3.82 per cent spiral reinforcement.

One set having 0.96 per cent spiral and 2.0 per cent vertical reinforcement.

One set having 0.96 per cent spiral and 4.0 per cent vertical reinforcement.

The tests were planned to throw light on the following important subjects: lateral and longitudinal deformations; effect of increasing the percentage of spiral steel; effect of length on plain as well as on spirally reinforced columns; the value of longitudinal as compared with spiral reinforcement.

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Series.	No.	Diam., in.	Area, sq. in.	L Length.	Ratio, L-D	Pitch, in.	Gage of Wire.	Nom. Diam. Wire, in.	Area Wire, sq. in.	Per Cent Column Section.	Column, lb. Total.	Column, lb. Per sq. in.	Manner of Failure.	Deformation .001″ Unit Load.	Deformation .001″ Unit Stress in Spiral.	Deformation .005″ Unit Load.	Deformation .005″ Unit Stress in Spiral.	Columns and Cylinders, days.	8x16″ Cylinders, Maximum Load, lb. Total.	8x16″ Cylinders, Maximum Load, lb. Per sq. in.
A	1	13.85	150.6	10′-0″	8.7	....	....	....	....	0	255 800	1698	Crushed 2 ft. from top	1625	....	....	....	79	90 000	1790
A	2	13.9	151.7	9′-11″	8.6	....	....	....	....	0	278 600	1836	Crushed near bottom	1800	....	....	....	74	89 100	1775
A	3*	14.6	167.4	9′-11.5″	8.1	....	....	....	....	0	226 300	1352	Crushed	1325	....	....	....	73	83 100	1655
Ave.	....	....	....	......	....	....	....	....	....	....	......	1767	....	1713	....	....	....	....	......	1740
B	4	14.0	153.9	10′-2″	8.7	1 1/16	No. 9	0.148	0.0172	0.46	511 300	3321	Crushed and buckled near center	2010	6 600	3300	81 000	78	140 000	2785
B	5	14.0	153.9	10′-1″	8.6	“	“	“	“	“	504 900	3280	Crushed; spiral broke as load fell	1770	10 300	3225	85 000	73	64 000	1273
B	6	14.0	153.9	10′-1″	8.6	“	“	“	“	“	525 500	3415	Crushed at center	2100	11 100	3300	85 000	73	92 400	1838
Ave.	....	....	....	......	....	....	....	....	....	....	......	3339	....	1960	....	3275	....	....	......	1965
C	7	14.0	153.9	10′-1.5″	8.7	1	No. 5	0.207	0.0337	0.96	709 300	4608	Crushed gradually till spiral broke	2075	10 200	3860	73 000	79	135 400	2693
C	8	14.0	153.9	10′-1″	8.6	“	“	“	“	“	706 800	4593	Spiral broke near center	1910	5 500	3770	60 000	73	106 200	2113
C	9	14.0	153.9	10′-4″	8.9	“	“	“	“	“	777 000	5049	Shear and compression	2125	2 730	4275	47 000	77	226 420	4504§
Ave.	....	....	....	......	....	....	....	....	....	....	......	4750	....	2037	....	3970	....	....	......	2403
D	10	14.0	153.9	10′-2″	8.7	1 1/8	5/16″	0.3125	0.0767	1.95	808 800	5256	Spiral broke 1 ft. from top	1500	1 960	3540	31 800	77	116 500	2318
D	11	14.0	153.9	10′-0.5″	8.6	“	“	“	“	“	832 300	5408	Spirals broke at top	1925	3 720	4290	24 700	73	93 000	1850
D	12	14.0	153.9	10′-4″	8.9	“	“	“	“	“	848 100	5511	No failure; general spalling	1975	2 930	4230	31 500	77	176 400	3510§
Ave.	....	....	....	......	....	....	....	....	....	....	......	....	....	1800	....	4020	....	....	......	2084
E	13	14.0	153.9	10′-3″	8.8	1 1/8	7/16″	0.4375	0.1503	3.82	848 200	5512	No failure; marked spalling	2100	2 380	4720	23 800	80	161 000	3202
E	14	14.0	153.9	10′-1″	8.6	“	“	“	“	“	848 100	5511	No failure; marked spalling	1715	3 690	4140	29 100	74	97 100	1932
E	15	14.0	153.9	10′-3″	8.8	“	“	“	“	“	847 900	5510	No failure; marked spalling	2100	2 650	4680	29 100	77	101 200	2013
Ave.	....	....	....	......	....	....	....	....	....	....	......	....	....	1972	....	4513	....	....	......	2382
F	16	13.9	151.7	4′-9″	4.1	....	....	....	....	0	310 300	2045	Crushed at center	1750	....	....	....	78	123 700	2461
F	17	14.2	158.4	5′-0″	4.2	....	....	....	....	0	338 500	2137	Crushed at center	1950	....	....	....	74	80 700	1605
F	18	14.0	153.9	5′-0″	4.3	....	....	....	....	0	321 200	2087	Crushed 1 ft. from bottom	1850	....	....	....	76	174 200	3465§
Ave.	....	....	....	......	....	....	....	....	....	....	......	2090	....	1850	....	....	....	....	......	2033
G	19	13.9	151.7	19′-6″	16.8	....	....	....	....	0	254 300	1676	Shear and compression near top	1650	....	....	....	78	88 700	1764
G	20	14.0	153.9	19′-4″	16.6	....	....	....	....	0	280 000	1819	Shear and compression 7 ft. from top	1810	....	....	....	79	136 850	2722
G	21*	14.2	158.4	19′-3″	16.3	....	....	....	....	0	169 700	1072	Crushed at center	....	....	....	....	77	117 000	2237
Ave.	....	....	....	......	....	....	....	....	....	....	......	1748	....	1730	....	....	....	....	......	2272
H	22	14.0	153.9	5′-1″	4.4	1	No. 5	0.207	0.0337	0.96	754 100	4900	Spiral broke near middle	1800	12 600	3470	83 000	82	......	....
H	23	14.0	153.9	5′-0.5″	4.3	“	“	“	“	“	765 300	4973	Spirals broke near top	2180	3 600	4080	55 000	77	129 150	2569
H	24	14.0	153.9	5′-2″	4.4	“	“	“	“	“	707 200	4595	Spirals broke near top	1875	9 660	3360	71 000	78	109 500	2178
Ave.	....	....	....	......	....	....	....	....	....	....	......	4823	....	1952	....	3637	....	....	......	2374
J	25	14.0	153.9	20′-0.63″	17.1	1	No. 5	0.207	0.0337	0.96	452 400	2939	Buckling	1850	4 830	....	....	78	......	....
J	26	14.0	153.9	20′-1″	17.2	“	“	“	“	“	554 300	3602	Buckling	1900	18 000	....	....	77	76 850	1529
J	27	14.0	153.9	20′-0″	17.1	“	“	“	“	“	656 500	4266	Buckling	2210	3 390	4235	27 500	76	140 600	2800
Ave.	....	....	....	......	....	....	....	....	....	....	......	....	....	1987	....	....	....	....	......	2165
K†	28	14.0	153.9	10′-1″	8.6	1	No. 5	0.207	0.0337	0.96	762 200	4952	Spiral broke as load was falling	2530	6 880	4450	56 000	82	152 750	3040
K†	29	14.0	153.9	10′-0″	8.6	“	“	“	“	“	826 700	5372	Spiral broke near top; spalling severe	2785	7 900	5025	72 700	78	......	....
K†	30	14.0	153.9	9′-10″	8.4	“	“	“	“	“	848 100	5511	No failure; spalling general	2590	12 600	4335	79 000	77	114 570	2280
Ave.	....	....	....	......	....	....	....	....	....	....	......	....	....	2635	....	4603	....	....	......	2660
L‡	31	14.0	153.9	10′-1″	8.6	1	No. 5	0.207	0.0337	0.96	712 100	4624	Spiral broke near top	2900	12 960	4370	66 500	77	101 600	2020
L‡	32	14.0	153.9	10′-1.75″	8.7	“	“	“	“	“	848 000	5510	No failure until re-loaded	2875	4 160	4810	63 000	80	178 800	3560§
L‡	33	14.0	153.9	9′-10″	8.4	“	“	“	“	“	848 000	5510	No failure	3400	8 780	5440	66 500	75	160 200	3190
Ave.	....	....	....	......	....	....	....	....	....	....	......	....	....	3058	....	4873	....	....	......	2605

* Columns 3 and 21 apparently abnormal; rejected from averages.
† These columns were reinforced with seven $\frac{3}{4}$-in. vertical rods having a total area of 3.09 sq. in., or 2 per cent of the column section.
‡ These columns were reinforced with fourteen $\frac{3}{4}$-in. vertical rods having a total area of 6.19 sq. in., or 4 per cent of the column section.
§ Abnormal cylinders rejected from averages.

Materials.

Cement.—The Portland cement used was of good quality, as shown by Table II, which gives typical results of tension and compression tests.

As determined by several tests, the average specific gravity of the cement was 3.15. The fineness tests showed that from 16.7 to 21.2 per cent was caught on a No. 200 sieve, and from 3.5 to 5 per cent on No. 100 sieve.

Table II.—Ultimate Strength of Cement and 1 : 3 (by Weight) Mortar, in Pounds per Square Inch.

Tensile Strength.				Compressive Strength.		Column Tests Corresponding.
Neat.		1 : 3 Mortar.		2-in. Neat Cubes.		
7 Days.	28 Days.	7 Days.	28 Days.	7 Days.	28 Days.	
695	767	235	274	9 770	11 060	Nos. 1 to 15 inclusive
636	653	218	260	9 760	11 250	do.
688	827	219	288	10 670	10 570	do.
Ave. 673	749	224	274	10 067	10 960	
790	786	182	406	4 540	8 038	Nos. 16 to 33 inclusive
908	864	249	364	5 165	10 792	do.
668	659	205	323	4 191	7 611	do.
726	773	210	310	.....	.....	do.
Ave. 773	771	212	351	4 632	8 814	

Sand.—The sand was from the Delaware River and of a variety commonly used locally in concrete. Its moisture content was about 5 per cent. Table III shows by the analysis of dry sand that the material was very fine.

Table III.—Typical Analysis of Dry Sand.

Size of Sieve.	Nominal Separation Size, in.	Percentage Passing Sieve.
10	0.0730	99.9
20	0.0335	94.5
30	0.0210	71.9
40	0.0155	29.3
50	0.0110	15.3
60	0.0092	10.2
70	0.0078	8.8
100	0.0055	3.4
150	0.00325	1.2
200	0.00275	0.8

Stone.—The crushed stone used for the larger aggregate was a very compact granite gneiss, locally but erroneously called Potsdam sand stone. This stone has nearly the hardness and strength of good trap rock or granite. It had 45 per cent voids, and was well graded; practically all passing a 1½-in. mesh, 60 per cent passing a 1-in. mesh and 3 per cent passing a ¼-in. mesh,

Concrete.—A 1 : 2 : 4 concrete was used for all columns, the required volumes being determined by weight based on 45 per cent voids in the larger aggregate, and 100 lb. per cu. ft. for cement. A Smith rotary hand mixer of about 3 cu. ft. capacity was used. The average weight of water was 8 per cent of the combined weights of cement, sand and stone. One test cylinder 8 in. in diameter and 16 in. long was made simultaneously with each column, the concrete for the cylinder being taken from a batch of column concrete, and each cylinder was tested in compression at the same age as its corresponding column.

Steel.—The lateral reinforcement in all columns consisted of high carbon steel spirals, the diameter of the wires being chosen to secure different percentages of lateral reinforcement. Table I shows the pitch of spirals, diameter of wires therein and corresponding percentages of steel for all columns.

Since these columns were molded with a very thin concrete veneer outside of the spirals, the percentage of vertical reinforcement was found by dividing the cross-sectional area of the vertical rods by that of the column

Table IV.—Tension Tests on Spiral Reinforcement.

Wire.	Actual Diameter, in.	Apparent Elastic Limit, lb. per sq. in.	Ultimate Tensile Strength, lb. per sq. in.	Modulus of Elasticity.
No. 9	0.149	70 000	140 000	29 800 000
No. 9	0.147	63 000	133 800	29 300 000
No. 5	0.204	71 000	134 500	26 400 000
No. 5	0.206	67 000	132 800	28 600 000
$\frac{5}{16}''$	0.310	67 000	93 600	29 200 000
$\frac{5}{16}''$	0.311	66 000	97 800	29 600 000
$\frac{7}{16}''$	0.438	70 000	99 100	28 600 000
$\frac{7}{16}''$	0.438	66 000	93 800	30 000 000

and multiplying by 100. It is simply the ratio of the area of steel to gross area of column.

For protection of steel against fire or corrosion, reinforced columns of course must have a thick veneer outside of the steel.

The percentage of spiral (lateral) reinforcement was obtained by dividing the volume of the spiral by the volume of column and multiplying by 100. This is most easily done by using the volumes of steel and column in one turn of the spiral. Thus, if p is the ratio of spiral reinforcement, or ratio of volume of steel to volume of column, and A_s one cross-sectional area of the spiral wire, a the pitch, r the radius of column, then $p=2\ A_s/ar$. On the percentage basis $p=200\ A_s/ar$.

Table IV shows results of tension tests on these spirals and in Fig. 1 are the stress-deformation curves for them, each curve representing the average of two tension tests. The table illustrates the effect of diameter on the ultimate tensile strength. The stress-deformation curves are of value in transforming deformations in spirals to corresponding unit stresses.

Fig. 2 shows the type of spiral reinforcement used. The spacing bars of the spirals were six small flats which are here neglected in working up results.

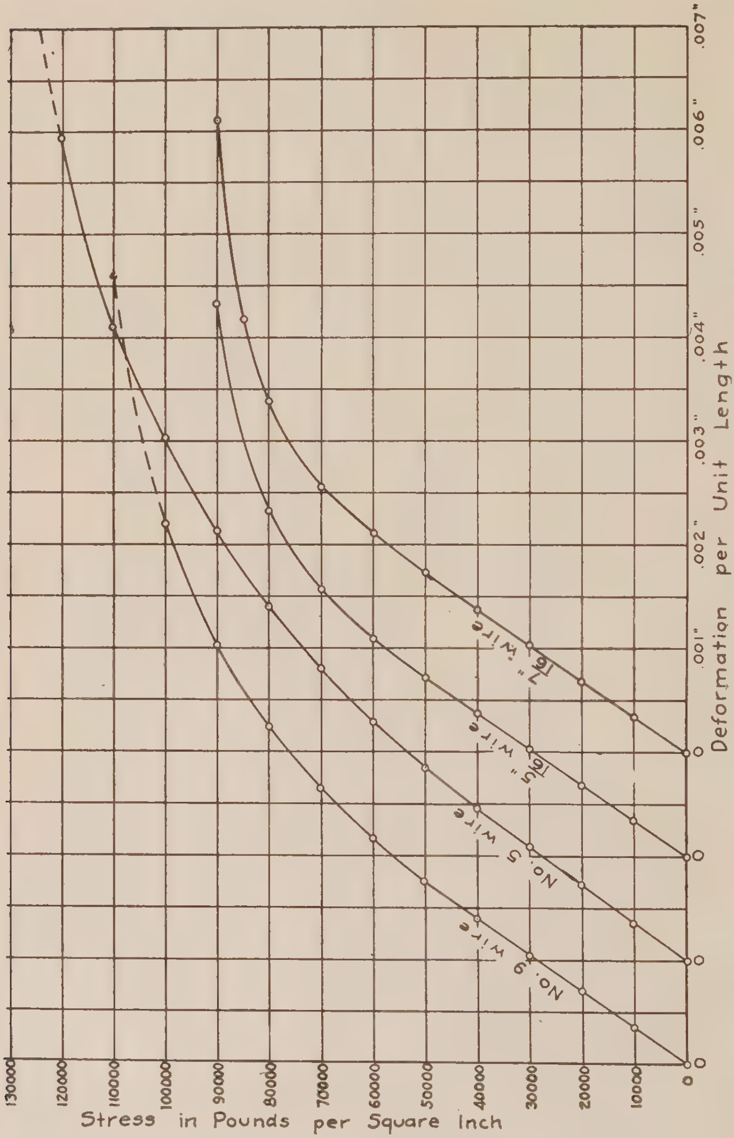

FIG. 1.—TENSILE TESTS OF SPIRAL WIRE.

Wherever vertical reinforcement was used it consisted either of seven or fourteen $\frac{3}{4}$-in. medium steel round rods in each column wired to the inside of the spirals. Tensile tests on these vertical rods are recorded in Table V. Fig. 3 gives the stress-deformation curves for tension and compression. Three compressive tests, made on specimens $3\frac{1}{2}$ in. long cut from the $\frac{3}{4}$-in.

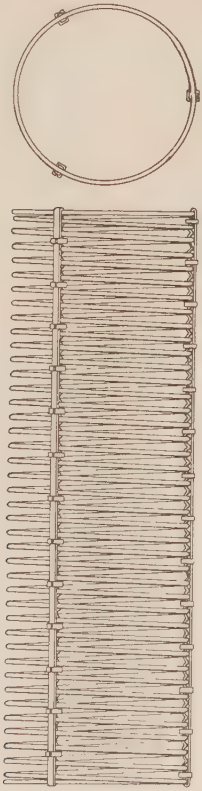

FIG. 2.—TYPE OF SPIRAL REINFORCEMENT.

vertical rods, are given in Table VI, with corresponding stress-deformation curves in Fig. 3.

Molding and Curing.

Columns and cylinders were molded in metal forms, precautions being taken to make all top and bottom bearing surfaces perpendicular to axes. The forms were removed in from 3 to 5 days, after which time the columns were sprinkled twice daily for one week, then allowed to remain standing

where cast till tested. Before being tested, each column was finished with a 1:1 mortar on its upper end. The temperature of the laboratory wherein all materials were stored and all columns made was quite uniform at about 70° F.

The cylinders were cured in damp sand after being removed from their forms.

Table V.—Tension Tests on $\frac{3}{4}$-In. Vertical Medium Steel Rods.

Actual Diameter, in.	Yield Point, lb. per sq. in.	Ultimate Tensile Strength, lb. per sq. in.	Modulus of Elasticity.
0.756	33 000	58 400	29 500 000
0.756	34 500	60 300	28 200 000
0.755	32 400	57 500	....
0.759	33 500	57 900	....
0.758	32 700	57 200	....

To permit reading lateral deformations directly on the steel spirals without cutting away the concrete veneer, and to obviate the inconveniences accompanying spalling of the veneer, all columns with lateral reinforcing were molded with the spirals practically in contact with the metal forms.

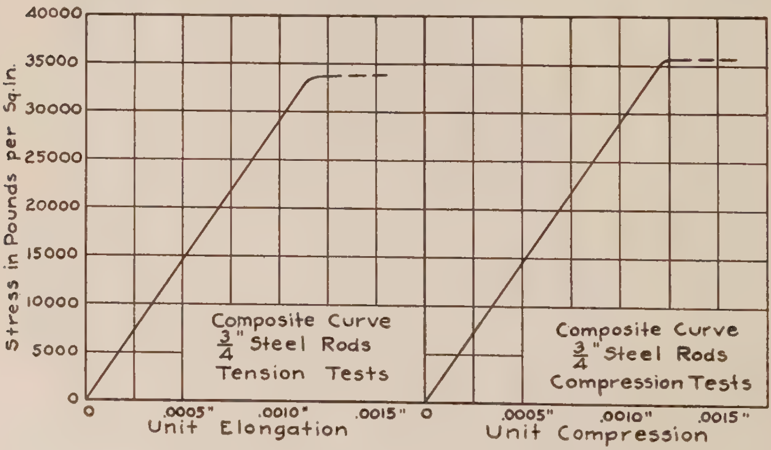

FIG. 3.—STRESS-DEFORMATION CURVES FOR VERTICAL STEEL.

This resulted in having the finished diameter of the concrete only a trifle, $\frac{1}{8}$ to $\frac{1}{4}$ in., larger than the exterior diameter of the spiral. At the same time there was sufficient bonding of the steel.

When tested the average age of the columns was 77 days, the maximum and minimum ages being 82 and 73 days.

Methods of Testing.

All columns were tested in compression in a vertical screw machine of 800,000 lb. capacity, the column resting directly on the weighing table, the load being applied on their upper ends through a spherical bearing block to secure uniform pressure. At the beginning of each test when the load reached about 50,000 lb. the effect of the spherical bearing block was overcome by two beveled annular rings and wedges and the column then had a square bearing at top as well as at bottom.

The loads reported in this paper are net applied loads and do not include the weights of columns.

The load increments of about 25,000 to 50,000 lb. were applied successively until the maximum load was reached, except in column 14. To this column increments of about 25,000 lb. were applied till a load of 450,000 lb. was reached, when the amount was decreased to 150,000 lb., then successively increased and decreased between these two limits till the longitudinal deformation showed practically no increase; then the load increments were continued till the maximum loading was reached.

Table VI.—Compression Tests on $\frac{3}{4}$-In. Vertical Medium Steel Rods.

Actual Diameter, in.	Ultimate Tensile Strength, lb. per sq. in.	Modulus of Elasticity.
0.752	32 500	28 500 000
0.753	35 500	30 500 000
0.759	35 500	28 000 000

Deformations.—Longitudinal and lateral deformations and center deflections were measured on all columns for the various loads.

Longitudinal deformations were determined by averaging readings from the Wissler dials, Fig. 4, attached at cardinal points on each column by means of steel yokes, the shortening of the column being communicated to the dials through wires fixed to other yokes near the opposite end of the column. The gage-length was 40 in. for 5-ft. columns, 80 or 100 in. for 10-ft. columns and 200 in. for those 20 ft. long.

Lateral deformation, that is, increases in diameter due to longitudinal compression, were measured by two specially constructed independent yokes, Fig. 4, arranged to give two readings at right angles to each other at the mid-height section of each column. These yokes, which were held by springs against the spirals of reinforced columns and against the concrete of plain columns, quadrupled the diametrical expansion by means of extension arms bearing Ames gages at their outer ends.

Center deflections were also determined in two directions at right angles to each other.

Results of Tests; Plain Columns.

Of the nine plain concrete columns three were practically 5 ft. long, three 10 ft. long, and three 20 ft. long. All were about 14 in. in diameter.

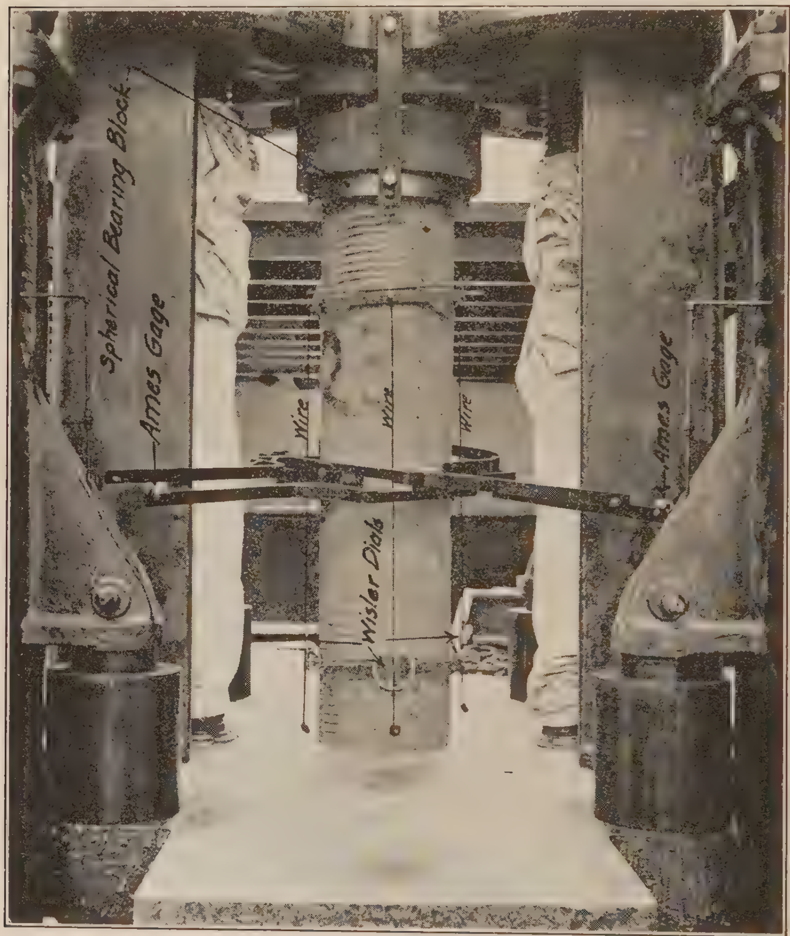

FIG. 4.—FIVE-FOOT COLUMN WITH LONGITUDINAL AND TRANSVERSE DEFORMATION YOKES.

The 20-ft. columns failed by shearing and compression; others by compression.

Ultimate Strength.—The average ultimate strength of cylinders corresponding to the plain columns was 2,174 lb. per sq. in. The average ultimate strengths of the plain columns were 2,090 lb. per sq. in. for 5-ft. columns,

1,767 lb. per sq. in. for 10-ft. columns, and 1,748 lb. per sq. in. for 20-ft. olumns. (See Table I.)

Effect of Length on Ultimate Strength.—The effect is shown by the results in the preceding paragraph. The average compressive strengths of

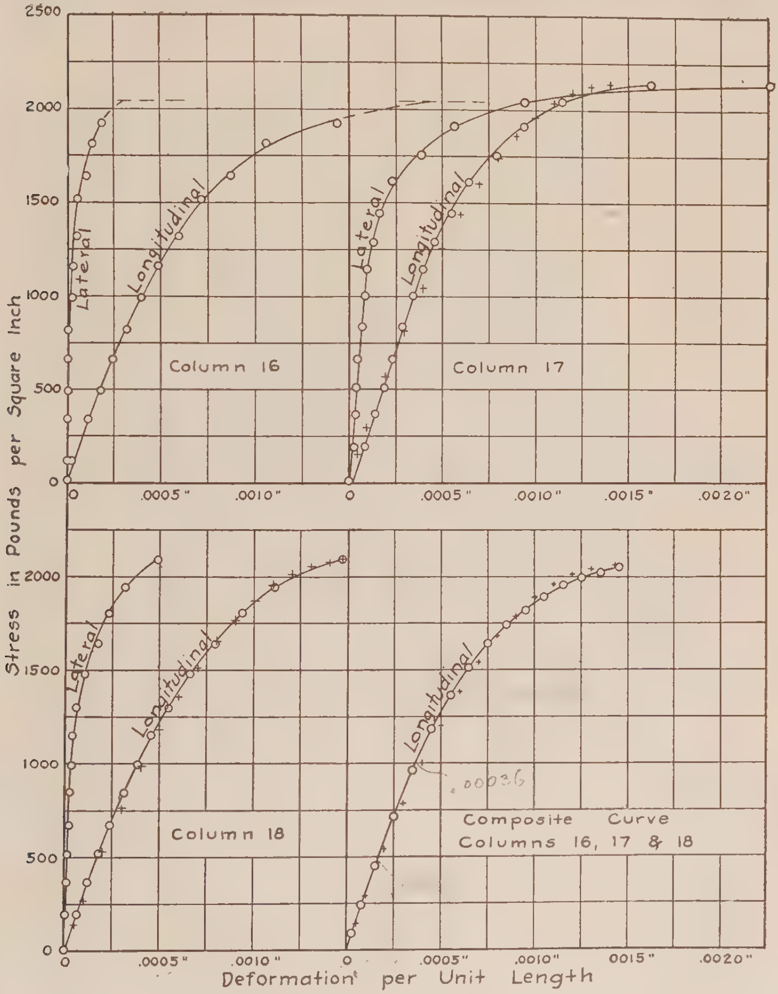

FIG. 5.—PLAIN CONCRETE COLUMNS 5 FT. LONG.

8 x 16-in. cylinders and 5-ft. columns 14 in. in diameter are very nearly equal, while the strengths of 10-ft. and 20-ft. columns are approximately equal, amounting to 81 per cent of the strength of cylinders. It is evident that sufficient tests have not been made to establish the proper relation between length and strength; nevertheless the results here presented are of value.

Longitudinal Deformation.—Fig. 5 shows individual stress-deformation curves for plain concrete 5-ft. columns, 16, 17 and 18, together with a composite curve for these three columns. The composite diagram and the diagrams for columns 16 and 18 are practically curved throughout while the

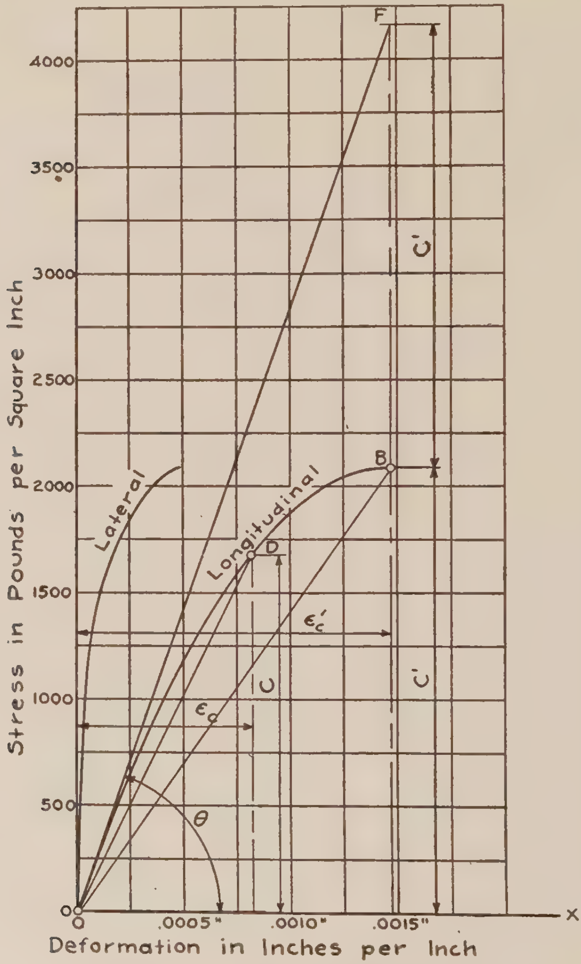

FIG. 6.—PARABOLIC STRESS-DEFORMATION CURVE.

diagram for column 17 is straight at its lower end to 1,000 lb. per sq. in.; then curved. Upon the composite stress-deformation curve and upon the curves for columns 17 and 18, Fig. 5, small crosses indicate points located on parabolas having vertices coinciding with maximum compressive stresses,

thus showing the close agreement between actual longitudinal deformations and those computed on the usual assumption of a parabolic stress-deformation curve.

The equation of the parabola as usually stated with respect to origin O, Fig. 6, is

$$C = E_c\mathrm{E}_c\left(1 - \frac{1\mathrm{E}_c}{2\mathrm{E}'_c}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where C is the compressive stress in pounds per square inch at any point, such as D, having a unit deformation of E_c; E_c is the initial modulus of elasticity; E'_c the unit deformation at maximum compressive strength.

Table VII.—Comparison of the Initial Modulus of Elasticity and the Modulus for the Maximum Unit Compression for Plain Concrete Columns.

Column No.	Length.	Maximum Unit Compression, lb. per sq. in.	Maximum Unit Deformation, in.	Modulus of Elasticity.	
				At Maximum Unit Compression.	Initial.
16	4′–9″	2045	0.002	1 020 000	2 700 000
17	5′–0″	2137	0.00162	1 319 000	3 100 000
18	5′–0″	2087	0.00147	1 420 000	2 950 000
Ave.	....	2090	....	....	2 920 000
1	10′–0″	1698	0.0012	1 415 000	2 100 000
2	9′–11″	1836	0.00115	1 597 000	2 550 000
3*	9′–10.5″	1352*	0.00129*	1 048 000*	3 200 000*
Ave.	....	1767	....	....	2 330 000
19	19′–6″	1676	0.00111	1 510 000	3 050 000
20	19′–4″	1819	0.0011	1 654 000	3 500 000
21*	19′–3″	1072*	0.0009*	1 191 000*	3 300 000*
Ave.	....	1748	....	....	3 275 000

* Rejected from averages.

The equation simply gives the unit compressive stress in terms of the corresponding unit deformation, the initial modulus of elasticity and the unit deformation at maximum compressive strength. For the three plain 5-ft. columns, the average initial modulus of elasticity is 2,920,000 lb. per sq. in.; the average of the maximum compressive strengths is 2,090 lb. per sq. in. Now since the unit deformation, E'_c at the maximum compressive strength, C', is equal to $2C'/E_c$, the average value of unit deformation at maximum compressive strength for these three 1 : 2 : 4 concrete columns is $2(2{,}090)/2{,}920{,}000 = 0.00143$.

Inserting 2,920,000 for E_c and 0.00143 for E'_c, equation (1) becomes

$$C = 2{,}920{,}000\,\mathrm{E}_c\left(1 - \frac{\mathrm{E}_c}{0.00286}\right)$$

This, then, is really the equation of the composite stress-deformation curves for the three 5-ft. columns with respect to the origin, and enables one to compute the compressive stress in pounds per square inch if the corresponding unit longitudinal deformation, E_c, is known.

Similarly the equation for the average of three 10-ft. plain columns is

$$C = 2{,}620{,}000\ E_c \left(1 - \frac{E_c}{0.00248}\right)$$

For the 20-ft. plain columns the equation is

$$C = 3{,}420{,}000\ E_c \left(1 - \frac{E_c}{0.00198}\right)$$

In this case, however, it should be remembered that this formula is based merely on the average unit compressive strength. For columns with large slenderness ratios, the average and maximum unit stresses near the maximum load must be very different.

Table VIII.—Young's Modulus for Plain Concrete Columns.

Length, ft.	Column No.	At 400 lb. per sq. in.	At 500 lb. per sq. in.	At $\frac{1}{3}$ Ultimate Strength.
5	16	2 700 000	2 700 000	2 700 000
	17	3 100 000	3 100 000	3 100 000
	18	2 900 000	2 860 000	2 800 000
10	1	2 100 000	2 100 000	2 100 000
	2	2 550 000	2 550 000	2 550 000
	3*	2 760 000*	2 700 000*	2 730 000*
20	19	2 920 000	2 860 000	2 800 000
	20	3 200 000	3 120 000	3 030 000
	21*	2 920 000*	2 700 000*	2 970 000*

* Rejected from all averages.

Modulus of Elasticity.—For materials such as concrete, where Hooke's Law does not hold, the modulus of elasticity may be obtained in any one of four ways; the resulting values being different.

First, the initial modulus is the slope of the stress-deformation curve at the origin; in other words, it is the tangent of the angle which the stress-deformation curve makes with the horizontal at the origin. Clearly this modulus of elasticity is the greatest possible, as may be readily seen from Fig. 6, where tan *FOX* is the initial modulus of elasticity. Furthermore, from the property of the parabola the initial modulus is just double the modulus for the point of maximum unit compressive stress, the latter being obtained by dividing this maximum unit compressive stress by its corresponding unit deformation, *i. e.*, $E_c = 2C'/E'_c$, Fig. 6. Table VII affords a comparison of the initial modulus and the modulus for the maximum unit compressive stress, showing that the relation just stated is fairly well sustained.

Second, Young's modulus of elasticity for a given point on the stress-

deformation curve is obtained by dividing the unit compressive stress for that point by its corresponding total unit deformation, *i. e.*, for point *D*, Fig. 6, the modulus is

$$C/\mathrm{E_c} = \tan BOX = E_c\left(1 - \frac{\mathrm{E_c}}{2\mathrm{E'_c}}\right)$$

For the point *B*, Fig. 6, Young's modulus is $C'/\mathrm{E'_c}$, or tan *BOX*, and this value is the smallest for any point on the curve between *O* and *B*. We have

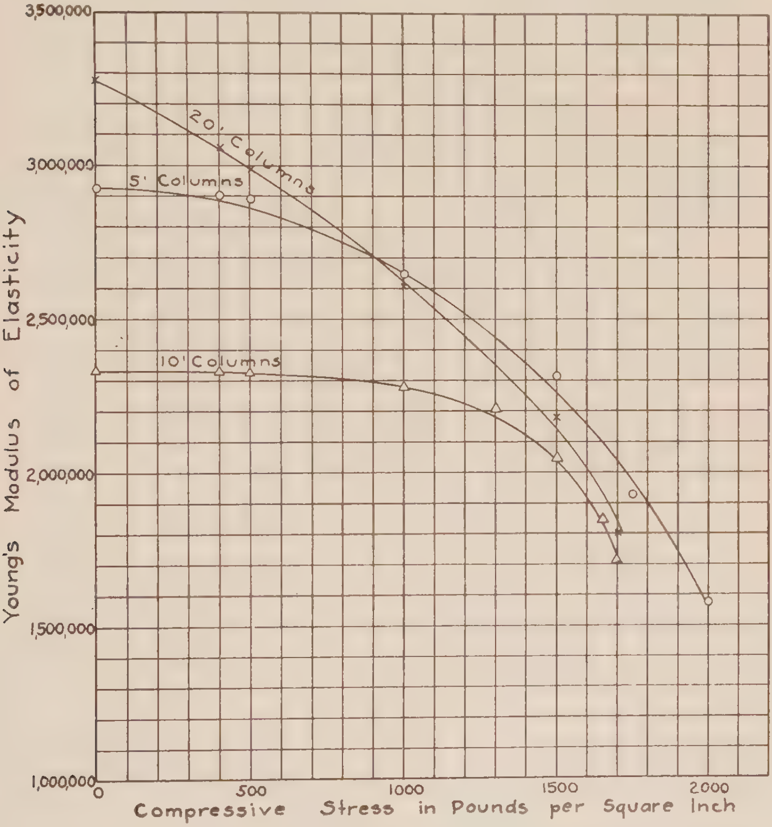

FIG. 7.—RELATION OF YOUNG'S MODULUS AND COMPRESSIVE STRESS.

stated, therefore, the two limits for Young's modulus, the minimum being for point *B*, tan *BOX*, and the maximum being the initial modulus, tan *FOX*. It thus appears that the inital modulus is merely the maximum limiting value of Young's modulus. Table VIII gives for plain columns Young's modulus for unit compressions of 400 and 500 lb. per sq. in., and for one-third the ultimate unit compressive stress, as determined from the tests.

Third, a modulus of elasticity for any loading may be obtained for the elastic deformation by dividing the unit stress, not by its corresponding total unit deformation, as in determining Young's modulus, but by the total unit deformation diminished by the permanent set. To secure the permanent sets it is necessary, from time to time during the test, to reduce the loading to zero. This method of obtaining the modulus is seldom used.

Fourth, the modulus of elasticity may also be defined for any point on the stress-deformation curve as the slope of the curve, that is, the slope of the tangent to the curve at the point in question. In this case the modulus of elasticity is the first differential coefficient of the stress-deformation curve,

TABLE IX.—POISSON'S RATIO FOR PLAIN COLUMNS.

Column No.	Length.	Poisson's Ratios at				
		500 lb. per sq. in.	1000 lb. per sq. in.	1500 lb. per sq. in.	2000 lb. per sq. in.	Maximum Compression.
16	4′–9″	0.010	0.065	0.070	.....	0.125
17	5′–0″	0.227	0.248	0.287	0.82	1.40
18	5′–0″	0.074	0.088	0.173	0.305	0.335
Ave.		0.104	0.134	0.177	0.563	0.620
1	10′–0″	.....	.....	.....	.....	.....
2	9′–11″	0.211	0.213	0.294	.....	0.336
3*	9′–10.5″	0.081	0.187	.....	.....	0.366
Ave.		.....	.....	.....	.....	.....
19	19′–6″	0.050	0.063	0.170	.....	0.250
20	19′–4″	0.100	0.155	0.123	.....	0.104
21*	19′–3″	0.025	0.015	.....	.....	0.132
Ave.		0.075	0.109	0.147	.....	0.177

* Columns 3 and 21 were rejected from all averages.

and the initial modulus above discussed is a special case of this fourth method. Differentiating equation (1) with respect to E_c as the variable, we have

$$\frac{dC}{dE_c} = E_c\left(1 - \frac{E_c}{E'_c}\right)$$

where dC/dE_c is the slope of the curve at a point having a unit deformation E_c. When $E_c = {}^\circ$, $dC/dE_c = E_c$, which is the initial modulus.

Unless otherwise stated, however, whenever the term modulus of elasticity is used in this paper Young's modulus is meant.

Fig. 7 shows the relation between the unit compressive stresses and corresponding moduli of elasticity for 5-ft., 10-ft. and 20-ft. plain columns. That the modulus decreases as the unit stress increases is clearly shown.

Lateral Deformation.—Under axial compression the diameter of a column is increased and its length decreased. This unit lateral deformation, *i. e.*, swelling resulting from longitudinal compression is, under low unit loads, much less than the unit longitudinal deformation. The ratio of unit

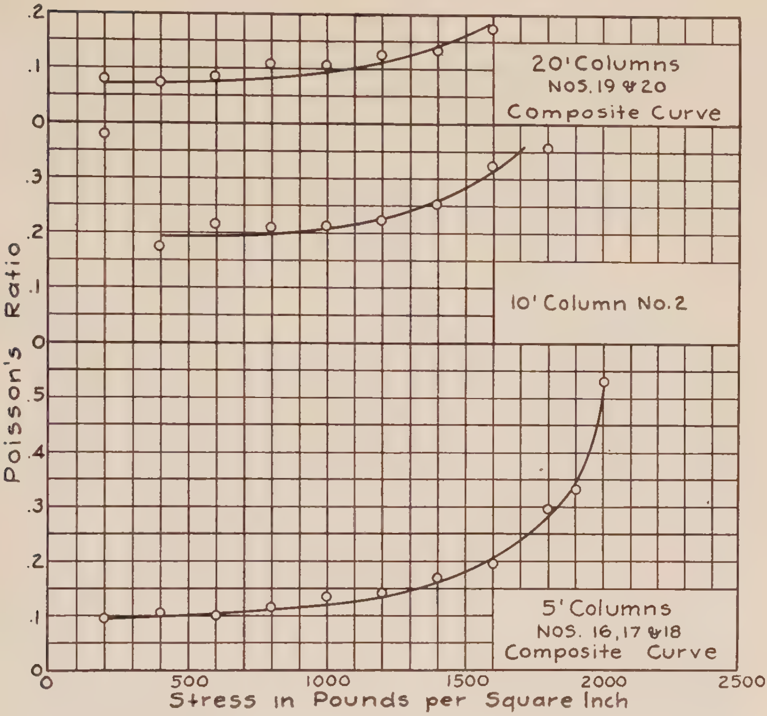

FIG. 8.—POISSON'S RATIO FOR PLAIN COLUMNS AS INFLUENCED BY COMPRESSIVE STRESS.

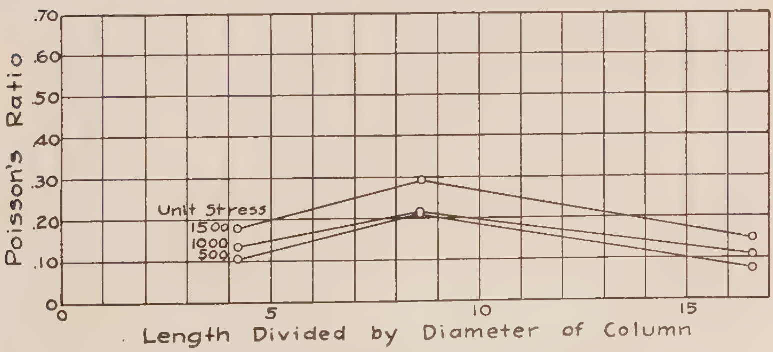

FIG. 9.—POISSON'S RATIO FOR PLAIN COLUMNS AS INFLUENCED BY SLENDERNESS RATIO.

lateral to unit longitudinal deformation is called Poisson's ratio. Hence, if the modulus of elasticity be constant for all directions, Poisson's ratio is the ratio between intensities of internal lateral tension and longitudinal compression. Referring to Fig. 5, it is seen that the unit lateral deformations increase somewhat after the same law as the unit longitudinal deformations, but in magnitude the former are much less than the latter. Thus, when the unit compression on column 18 was 500 lb. per sq. in., the corresponding longitudinal shortening per inch of length was 0.000175 in., while the unit

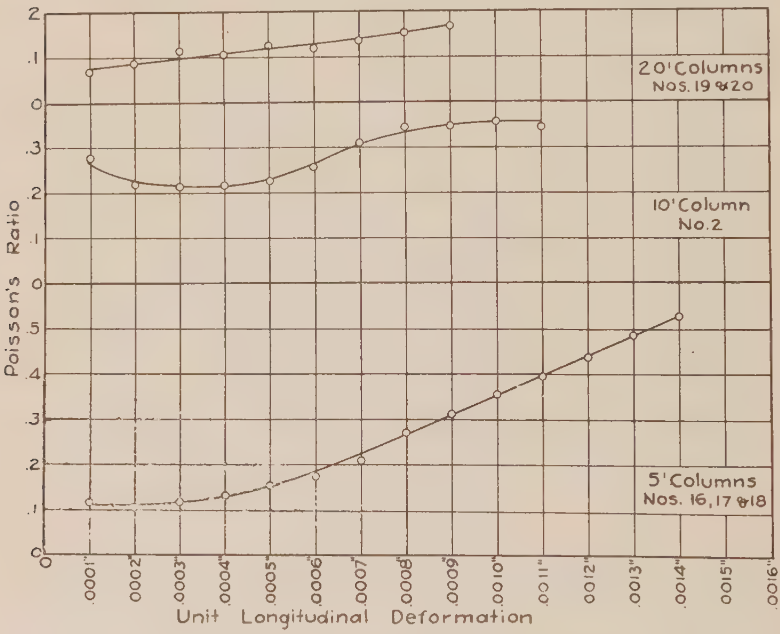

FIG. 10.—POISSON'S RATIO, PLAIN COLUMNS, AS INFLUENCED BY LONGITUDINAL DEFORMATION.

lateral deformation, *i. e.*, lateral deformation per inch of diameter, was 0.000013 in. In this case, Poisson's ratio is 0.000013 in. divided by 0.000175 in. or 0.074; and in a similar manner Poisson's ratio may be computed for various longitudinal unit compressions, as recorded in Table IX.

Fig. 8, showing the relation between Poisson's ratio and corresponding unit compressive stress, indicates that as the unit stress increases Poisson's ratio likewise increases. Fig. 9 illustrates the effect of length, and Fig. 10 shows how Poisson's ratio increases as the unit longitudinal deformation increases. This is especially marked in the curve for 5-ft. columns.

Results of Tests; Reinforced Columns.

Of the 24 reinforced columns, three were 5 ft. long, eighteen 10 ft. long and three 20 ft. long. All were about 14 in. in diameter. Table I gives the necessary data as to kinds and percentages of reinforcement.

During the application of the loads, the first visible effect of compression was the buckling of the outer straps of the spacing bars, which began at unit compressive stresses slightly above the ultimate strength of plain concrete. This was later followed, at considerably higher unit stresses, by spalling of the concrete. Except for the 20-ft. columns which buckled nearly 2 in.,

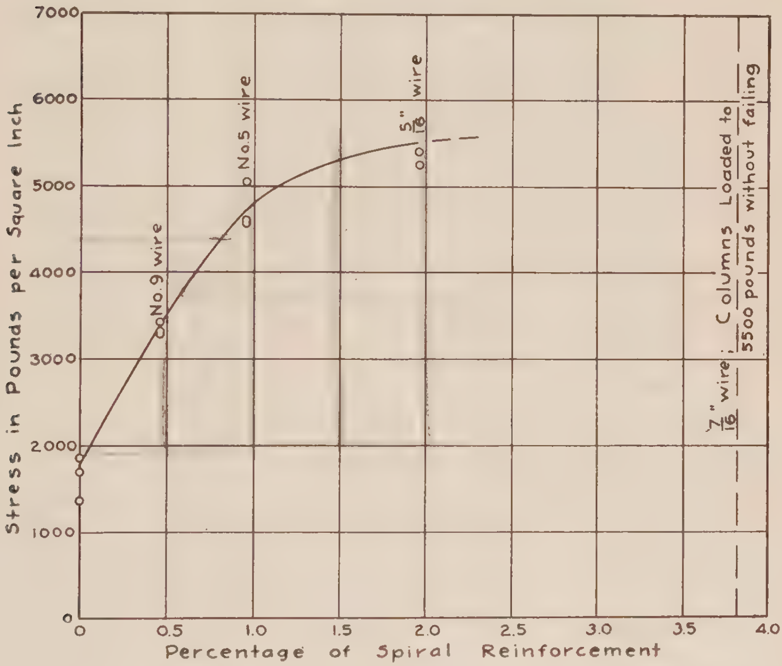

FIG. 11.—EFFECT OF AMOUNT OF SPIRAL ON ULTIMATE STRENGTH.

failure in almost all cases was accompanied by breaking of spirals shortly after the maximum load had been reached.

Effect on Ultimate Strength of Spirals of Various Percentages.—Spiral reinforcement increases the ultimate strength, practically in proportion to the percentage of the spirals until 1 per cent is reached. For percentages larger than 1, the rate of increase is not so great. The effect of spirals is shown by the fact that although the average compressive strength of plain 10-ft. columns of Group A is 1,767 lb. per sq. in., the average for 10-ft. columns of group B, having 0.46 per cent spiral, is 3,339 lb. per sq. in., an increase of 89 per cent; for 0.96 per cent reinforcement columns of group C the

increase is 169 per cent over plain columns; for 1.95 per cent reinforcement columns of group D the increase over plain columns is 205 per cent. Fig. 11 shows this effect. It is seen that the increase in strength from plain to 0.46 per cent lateral reinforcement is about the same as the increase from 0.46 to 0.96 per cent. In this connection it should be borne in mind that the

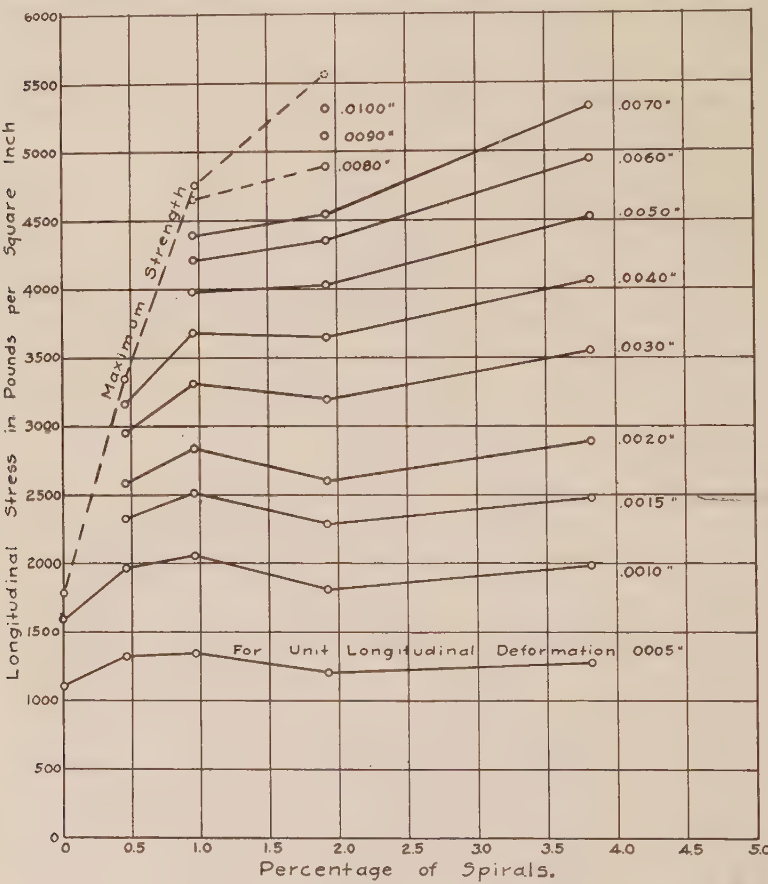

FIG. 12.—EFFECT OF DIFFERENT SPIRALS ON 10-FT. COLUMNS.

ultimate tensile strengths of wires of larger diameters are less per square inch than those of wires of smaller diameters.

The columns of group E, reinforced with $\frac{7}{16}$ in. diameter spirals, resisted without failure the full strength of the testing machine, but their behavior indicated much distress under the maximum applied load.

Effect of Spirals of Various Percentages on Longitudinal Deformation and Modulus of Elasticity.—For the lower unit compressive stresses, say under

2500 lb. per sq. in., longitudinal deformations are only very slightly affected by spiral reinforcement, as shown in Fig. 12, but under large unit compressive stresses the spirals restrain the lateral swelling of the concrete, hence, retard the longitudinal deformations. Thus we see that a unit compressive stress of 2800 lb. per sq. in. was necessary to produce a unit longitudinal deformation of 0.0020 in. for group C having 0.96 per cent spirals, whereas for plain columns the ultimate strength of 1770 lb. per sq. in. produced unit deformations of only about one-half of 0.0020 in. So far as longitudinal deformation is concerned, it is evident that until a compressive stress of about 2750 lb. per sq. in. was reached, the effect of the spirals was nearly constant, whether 0.46 or 3.82 per cent were used; in other words, only a small percentage of spiral was needed to slightly restrain the concrete, and practically nothing was gained, so far as longitudinal deformations are concerned, by using additional lateral steel. Under the larger unit compressive stresses of, say, 3500 to 4500 lb. per sq. in., 0.96 per cent spirals are more effective than 0.46 per cent spirals in restraining longitudinal deformation, but 1.95 per cent

Table X.—Moduli of Elasticity of Ten-Foot Columns, Plain and Reinforced with Spirals.

Group.	Lateral Reinforcement, percentage.	Moduli of Elasticity in Pounds per Square Inch.				
		At ½ Ultimate Strength.	At 500 lb. per sq. in.	At 1000 lb. per sq. in.	At 2000 lb. per sq. in.	At 3000 lb. per sq. in.
A	0	2 330 000	2 330 000	2 280 000	....	....
B	0.46	3 020 000	3 330 000	2 890 000	1 920 000	950 000
C	0.96	2 760 000	3 290 000	2 900 000	2 070 000	1 290 000
D	1.95	2 200 000	3 090 000	2 580 000	1 710 000	1 170 000
E	3.82	....	3 080 000	2 840 000	1 970 000	1 400 000

has practically no advantage over 0.96 per cent. (See Fig. 12.) Of course, the greater the percentage of steel the greater was the ultimate strength.

The moduli of elasticity for longitudinal deformations for 10-ft. plain columns, group A, and for 10-ft. columns reinforced with spirals only, groups B, C, D and E, are given in Table X. The effect of spirals on the modulus of elasticity should be studied in connection with the remarks of the preceding paragraph.

Effect of Various Percentages of Spiral Steel on Lateral Deformation.—The remarks under this heading apply to columns reinforced with spirals only. When a spirally-reinforced column is subjected to compressive stresses, the longitudinal shortening is accompanied by a transverse increase in diameter of the column, and if a bond of union exist between the spiral and the concrete, as it should, the transverse swelling of the concrete must produce immediate tension in the spiral. Inasmuch, however, as Poisson's ratio under low unit stresses is very small, as shown for plain columns in Fig. 8, it is evident that under small longitudinal stresses the lateral deformations and their corresponding stresses in the spiral are small. Now, as the longi-

tudinal compressive stress is increased, Poisson's ratio is also increased and this is accompanied by increased tensile stresses in the spiral. As the load upon the column increases, a point is finally reached where the compressive strength of the concrete is overcome and the concrete disintegrates. We then have a mass of disintegrated material held in place by the spiral, if the spiral

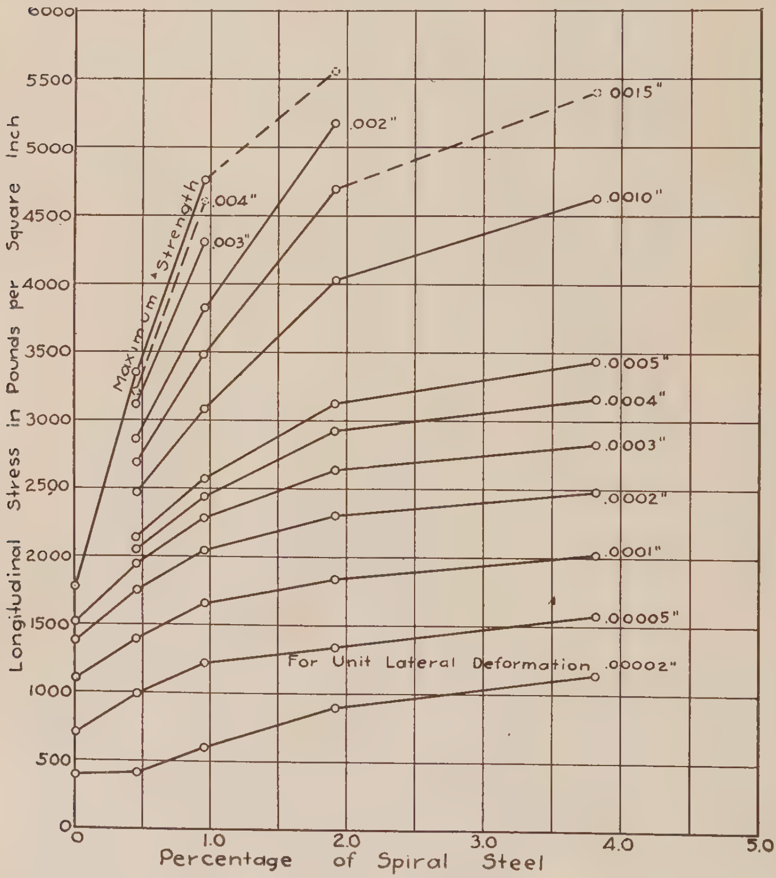

FIG. 13.—EFFECT OF DIFFERENT SPIRALS ON LATERAL DEFORMATION.

has not up to this time reached its ultimate strength. It is quite evident, therefore, that a large percentage of spiral steel exerts a very considerable restraining influence on the lateral deformation of the column.

In Fig. 13 all of these effects are clearly illustrated. In the first place, as one would expect, the general shapes of curves in Fig. 13 are somewhat similar to those of Fig. 12 where the relation between longitudinal stresses

and longitudinal deformations are shown. Fig. 13 indicates clearly that spirals as small as 0.48 per cent restrain lateral deformation, but that under very low unit compressive stresses these small spirals have only little effect. It also shows that from very small unit lateral deformations to very large ones the spiral exerts a very marked restraining influence. For example,

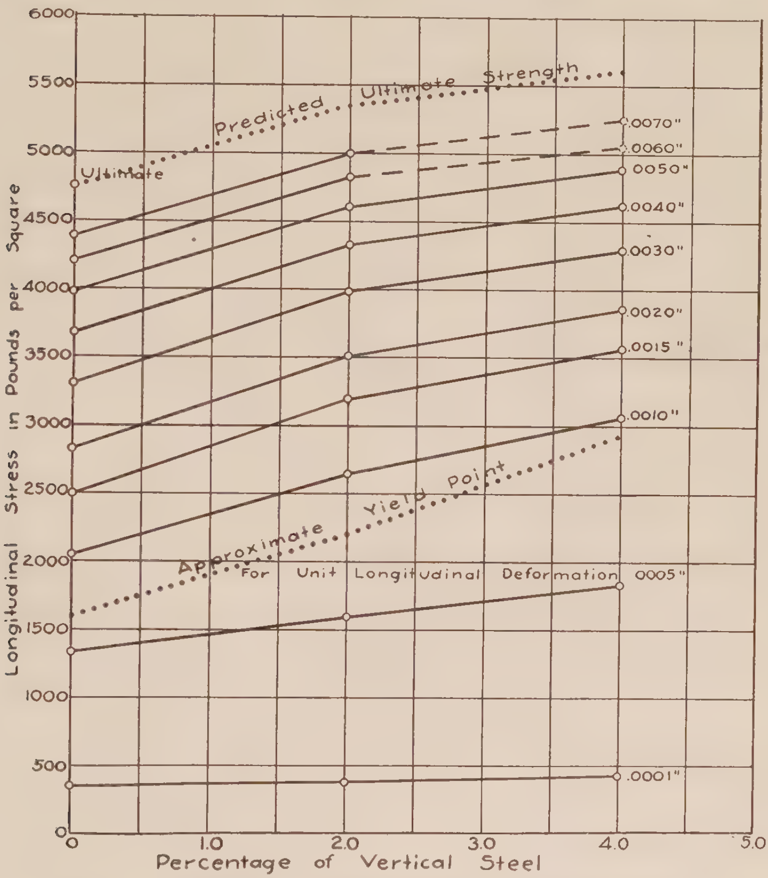

FIG. 14.—EFFECT OF DIFFERENT VERTICAL REINFORCEMENT.

consider the unit lateral deformation of 0.00002 in. per in. In columns with no spirals, less than 500 lb. per sq. in. are required to produce this deformation, whereas in columns with nearly 4 per cent spiral steel about 1150 lb. per sq. in. are required to produce the same unit deformation. Furthermore, it should be noticed how much more inclined are the left portions of the curves corresponding to the larger deformations; indicating that as the

compressive stress reaches the ultimate strength of plain concrete the spiral exerts a much greater influence. A comparison of the slope of the curve for 0.0003 in. with the slopes of the left portions of the curves below it shows this feature quite clearly. Under large unit compressive stresses the curves for columns having from 0.5 to 1 per cent of spiral steel show to a marked degree the effect of increasing the size of the spiral. Perhaps the most

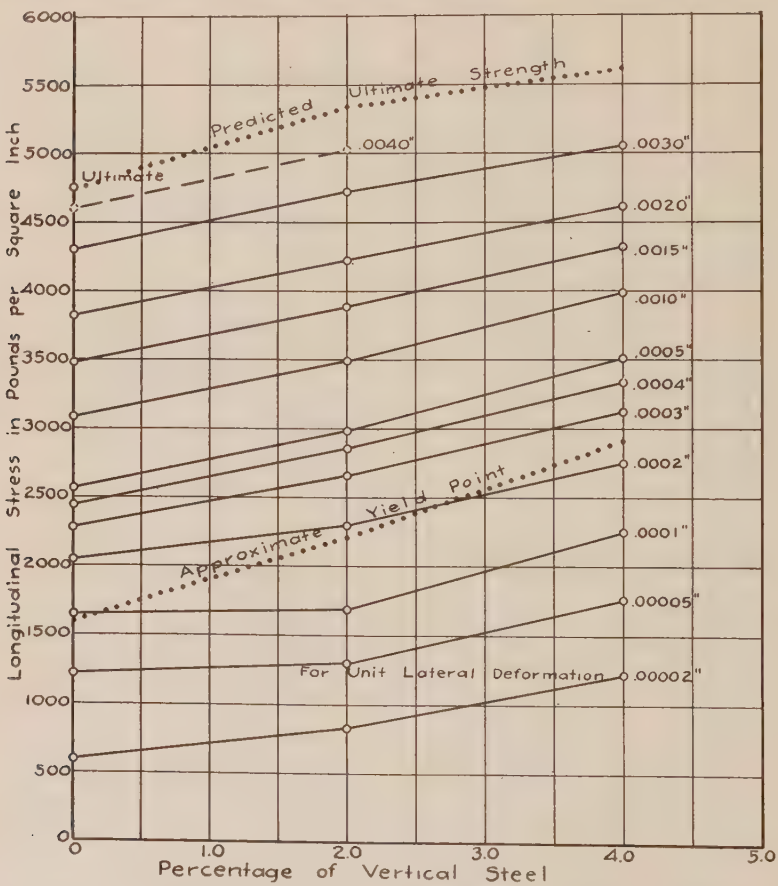

FIG. 15.—EFFECT OF DIFFERENT PERCENTAGES OF VERTICAL STEEL.

important point brought out by the curves of Fig. 13 is that the spirals of 1 per cent or more are most effective, and that spirals of only 0.5 per cent exert very little influence under working unit stresses.

Effect of Various Percentages of Vertical Steel on Longitudinal Deformation.—It is here assumed that the longitudinal steel is placed within the spiral, so that it is restrained from buckling outwardly by the spiral, and

from buckling inwardly by the concrete. For spirally reinforced columns, the effect of increasing the vertical steel is to decrease the longitudinal deformation, because the modulus of elasticity of vertical steel is materially greater than the modulus of elasticity of the concrete. The modulus being larger, the deformation corresponding to a given stress per square inch is smaller.

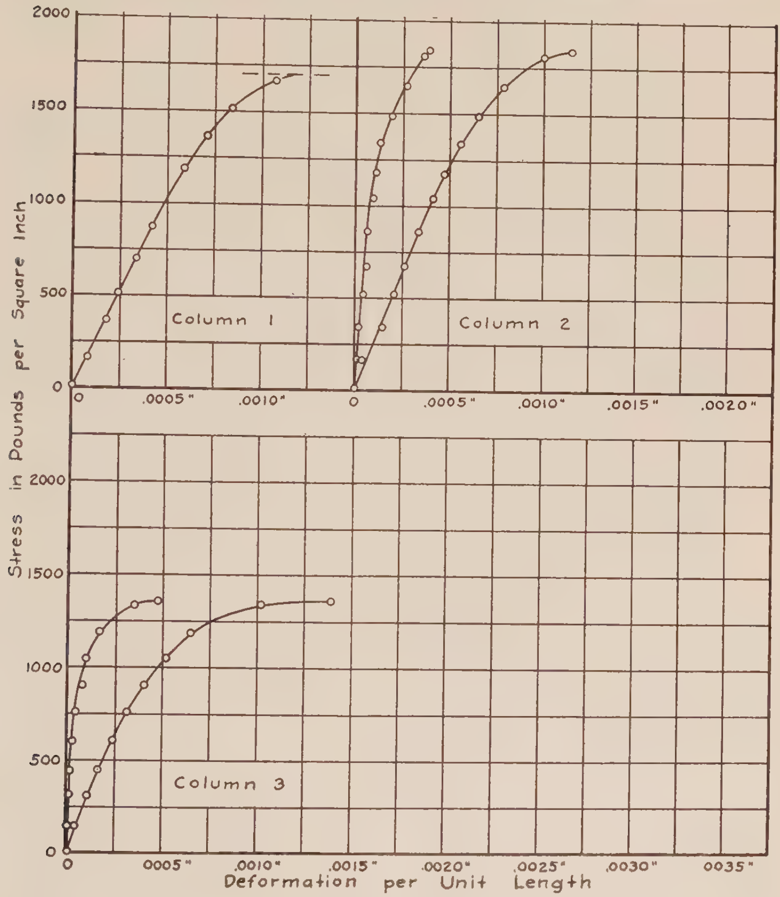

FIG. 16.—PLAIN CONCRETE COLUMNS 10 FT. LONG.

Referring to Fig. 14, it is seen that the effect of the vertical steel is quite marked for larger unit compressive stresses. While it takes 4000 lb. per sq. in. compressive stress, for example, to produce a unit longitudinal deformation of 0.005 in. with no vertical steel present, it takes a unit stress of about 4800 lb. per sq. in. to produce the same longitudinal unit deformation in a column containing 4 per cent vertical steel. To produce a unit longitudinal

deformation of 0.0005 in. about 1350 lb. per sq. in. are required in a column with no vertical steel whereas in one with 4 per cent vertical steel about 1800 lb. per sq. in. are required. Under small unit compressive stresses the effect of the vertical steel is not relatively great. This is shown in Fig. 14 by the lowest line, which indicates the unit longitudinal deformations of 0.0001 in. per in. This line slopes only slightly upward to the right, showing that increasing the percentage of longitudinal steel increases the resistance to longitudinal deformation. In studying Fig. 14 and the above remarks,

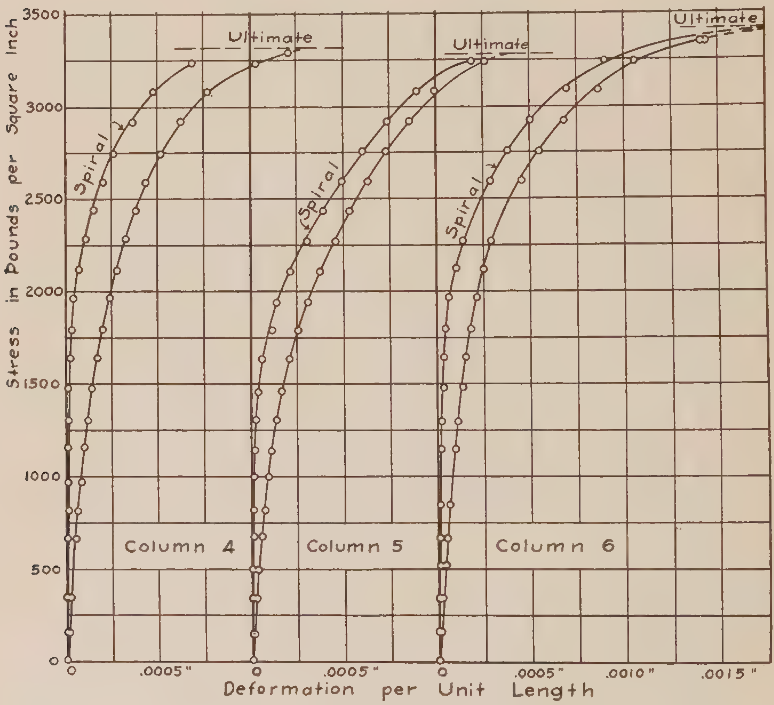

FIG. 17.—COLUMNS 10 FT. LONG, ½ OF 1 PER CENT SPIRAL.

it should be borne in mind that the spiral reinforcement is constant in all cases included in the diagram. Each column had 1 per cent spiral reinforcement, so that the only variable in the physical makeup of the column was the percentage of vertical steel.

Effect of Various Percentages of Vertical Steel on Lateral Deformation.—This effect is clearly shown in Fig. 15, in which the two variables are longitudinal stress in pounds per square inch and percentages of vertical steel. All columns used in plotting of the curves of Fig. 15 practically had 1 per cent spiral. The figure speaks for itself.

Effect of Vertical Reinforcement on Ultimate Strength and Yield Point.—It is unfortunate that the ultimate strength of several columns having vertical steel exceeded the capacity of the 800,000-lb. testing machine. There

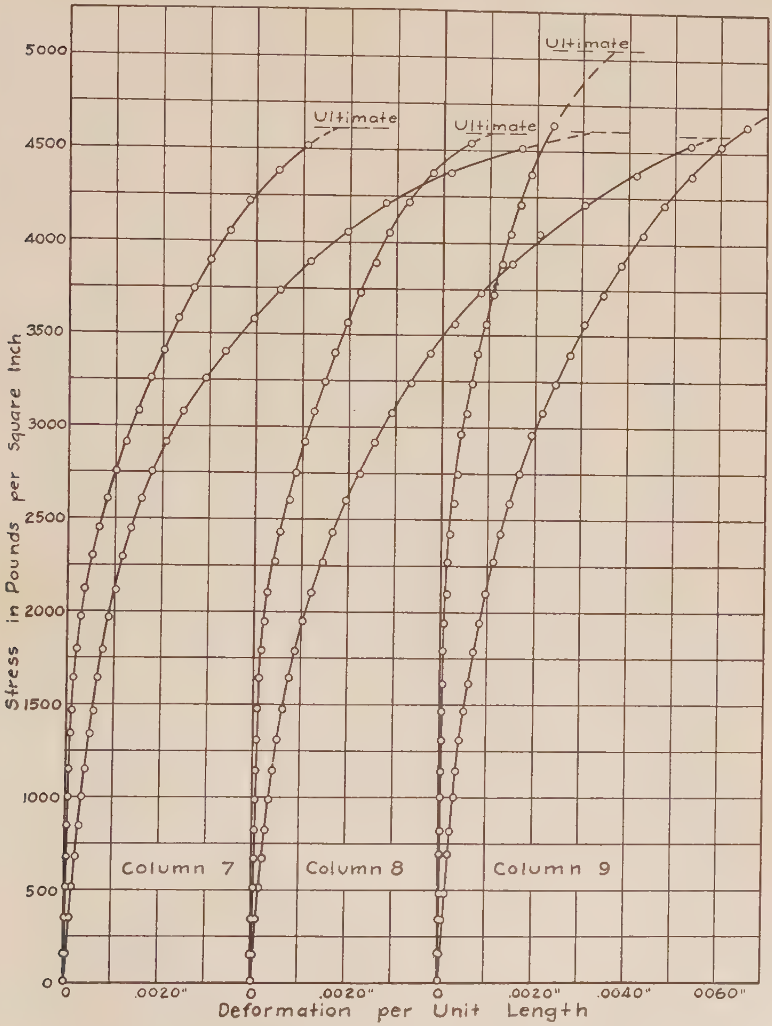

FIG. 18.—COLUMN 10 FT. LONG, 1 PER CENT SPIRAL.

appears to be sufficient data to predict with reasonable accuracy what the ultimate strength would have been. In Figs. 14 and 15 the predicted ultimate strengths are plotted. From the data at hand, about all that can be

said is that the trend of these curves in Figs. 14 and 15 indicates that vertical steel increases the ultimate strength and that the greater the percentages of

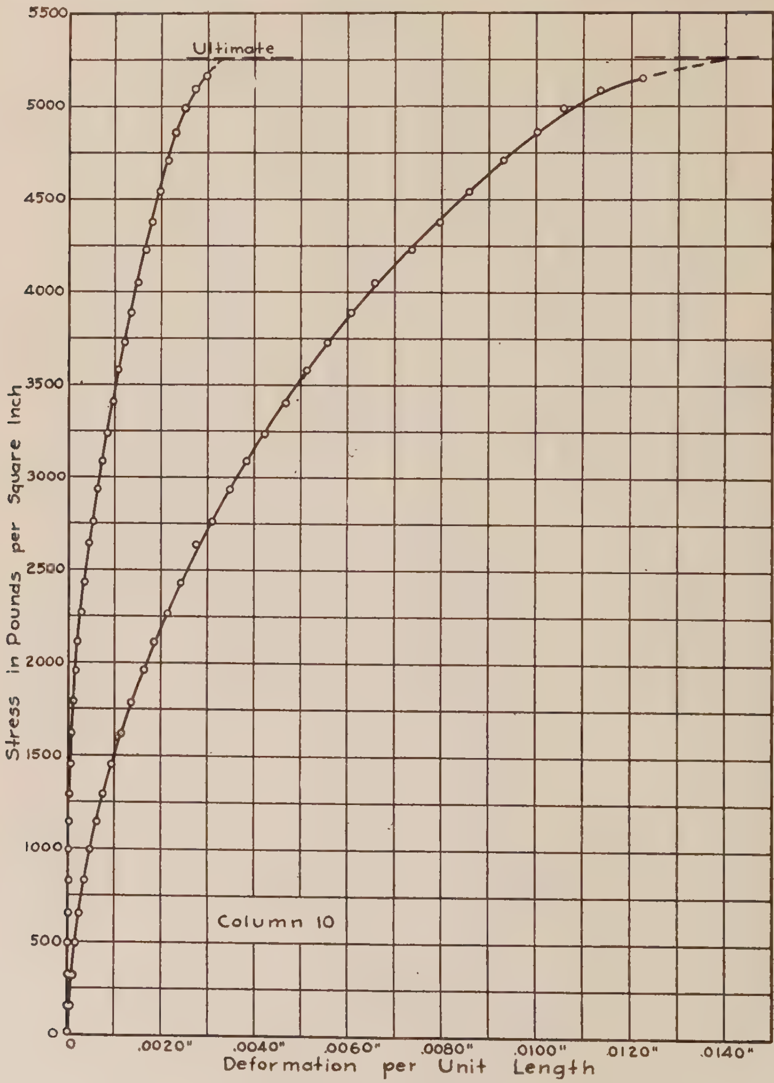

FIG. 19.—COLUMN 10 FT. LONG, 2 PER CENT SPIRAL.

vertical steel the greater the ultimate strength. This applies only to columns in which the vertical rods are properly supported against buckling.

Figs. 14 and 15 also show curves representing the approximate value of

the yield points of columns with 1 per cent spiral and with varying percentages of vertical steel. Here again one observes that as the percentage

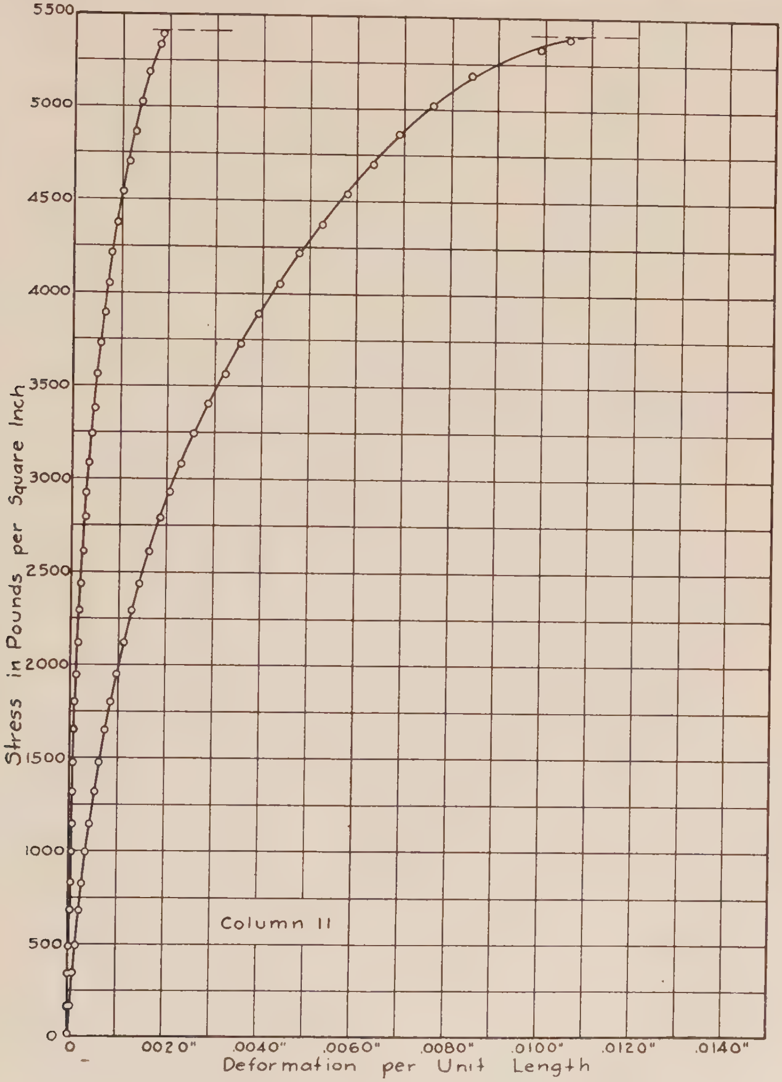

FIG. 20.—COLUMN 10 FT. LONG, 2 PER CENT SPIRAL.

of steel increases the yield point of the column also increases. Referring to Fig. 14; in a spiral column with no vertical steel the yield point is about 1500 lb. per sq. in., from which it increases quite uniformly as the percentage

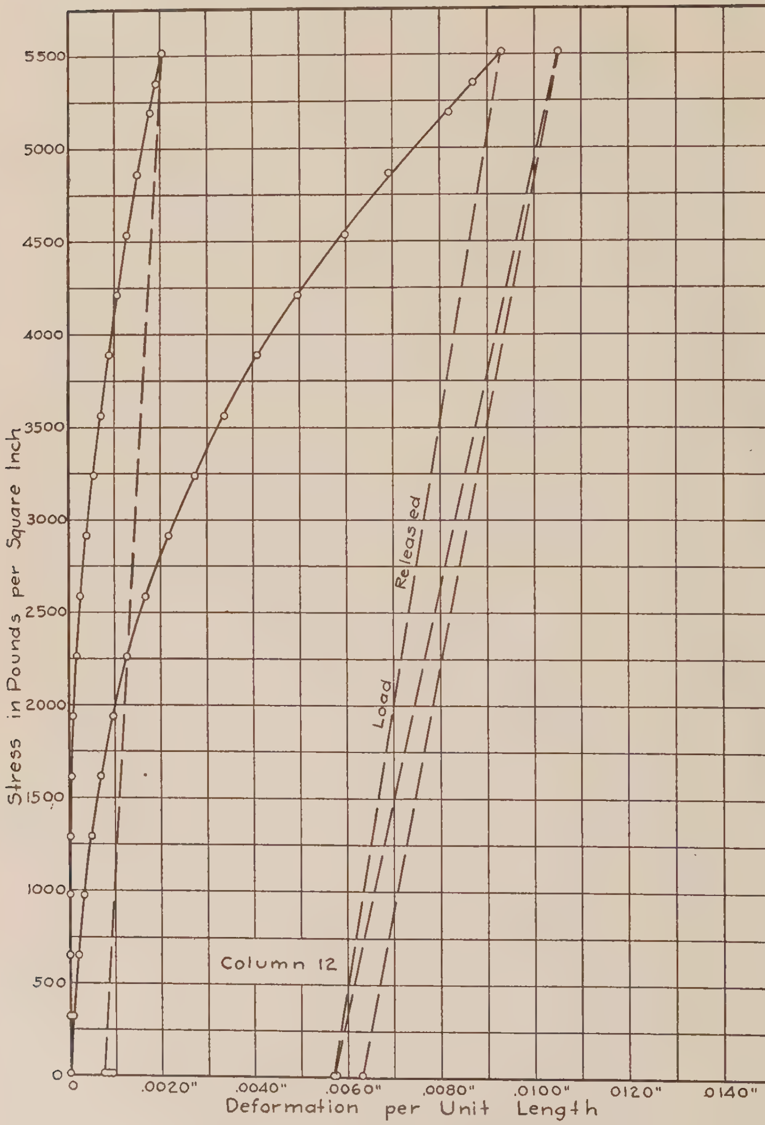

FIG. 21.—COLUMN 10 FT. LONG, 2 PER CENT SPIRAL.

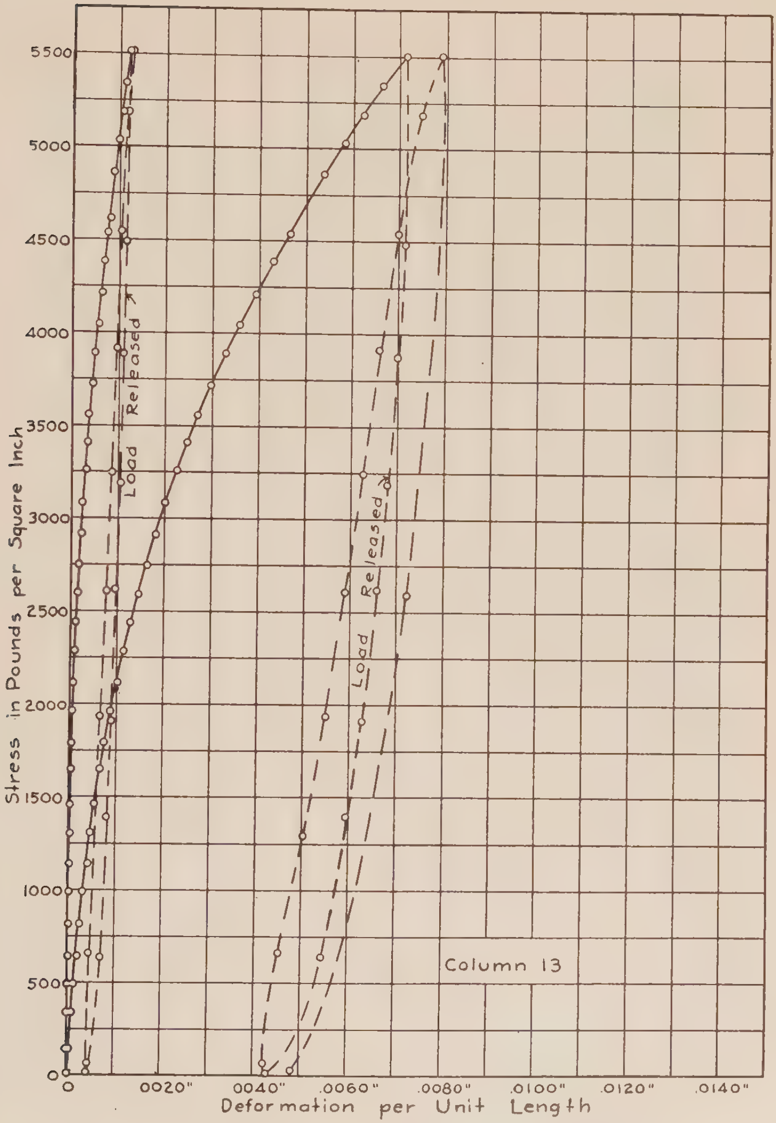

FIG. 22.—COLUMN 10 FT. LONG, 4 PER CENT SPIRAL.

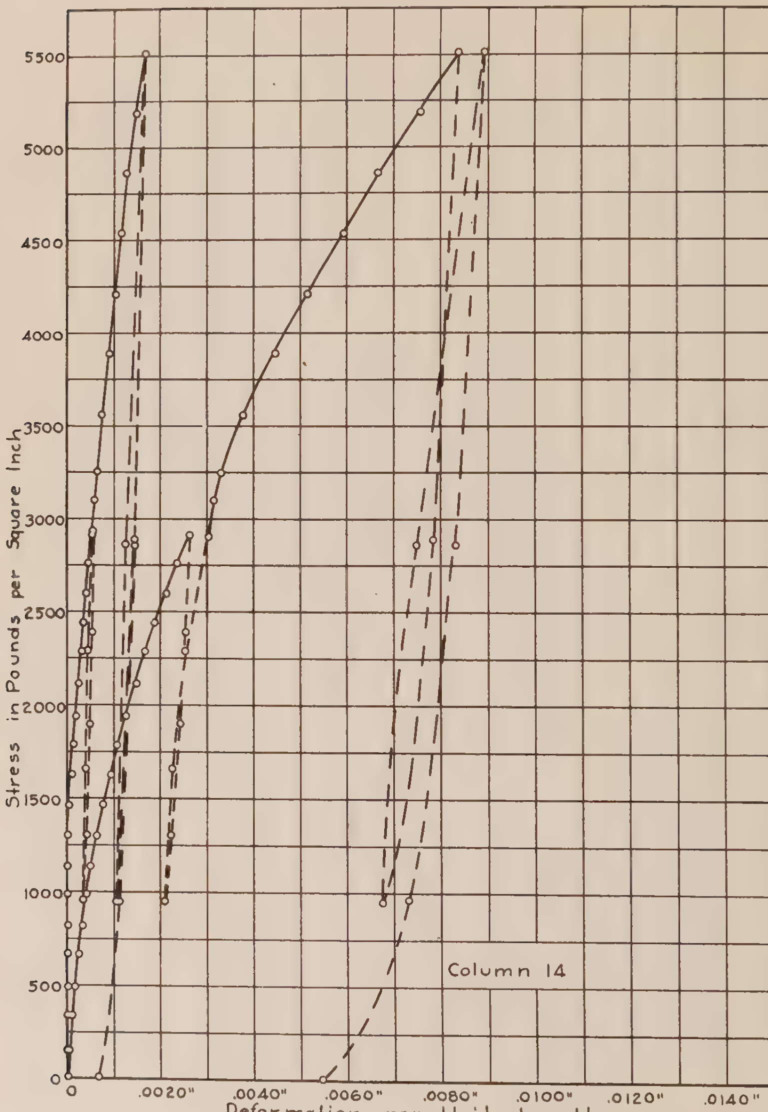

FIG. 23.—COLUMN 20 FT. LONG, 4 PER CENT SPIRAL

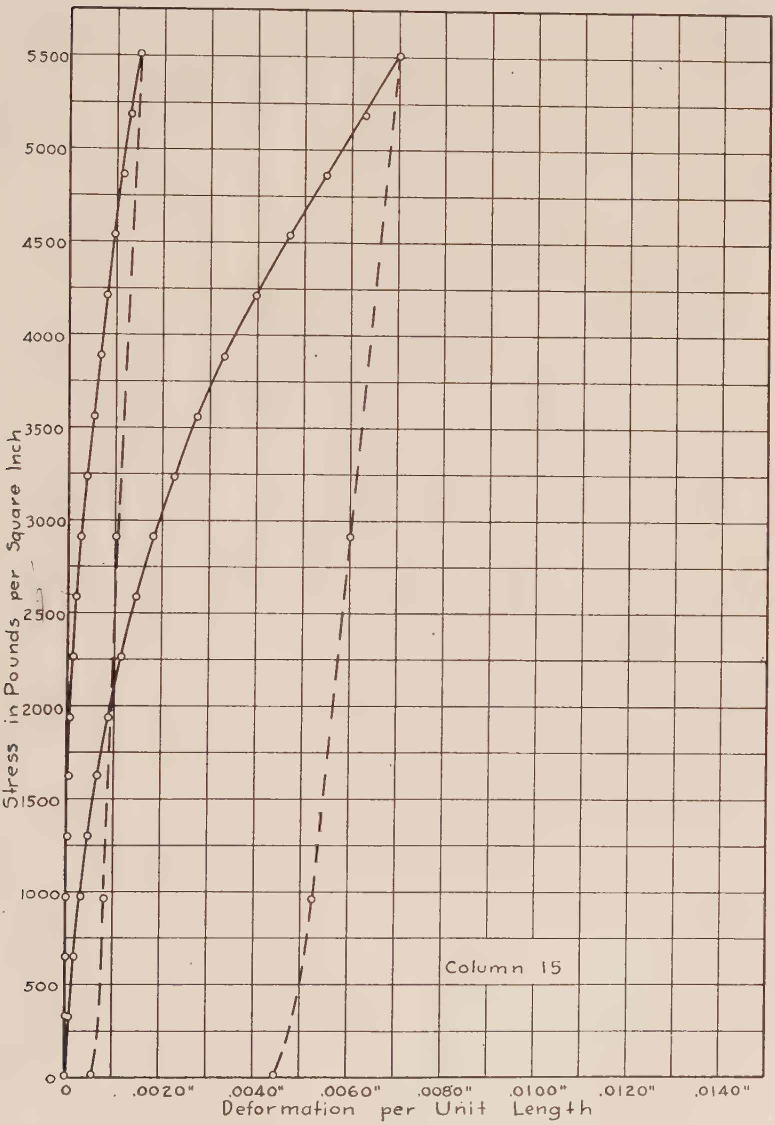

FIG. 24.—COLUMN 10 FT. LONG, 4 PER CENT SPIRAL.

of vertical steel increases until for a 1 per cent spiral column with 4 per cent vertical steel, the yield point is nearly 3000 lb. per sq. in. It perhaps goes without saying that this curve representing yield points cannot be located with great precision. Should Johnson's apparent elastic limit be taken, or should we

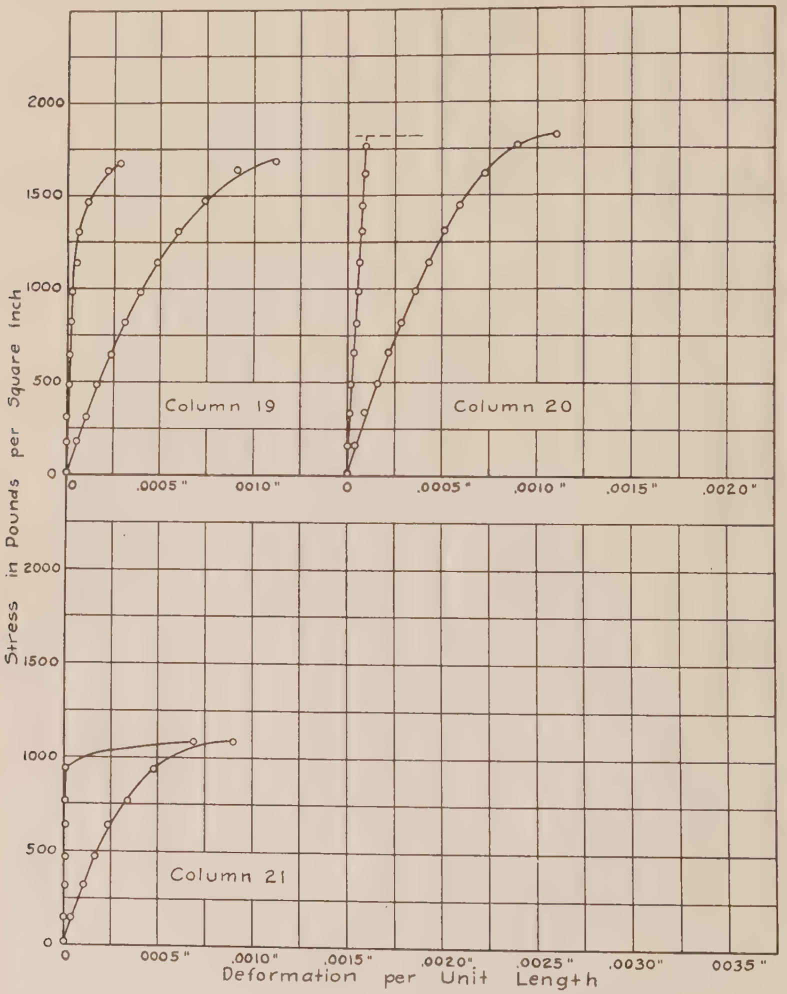

FIG. 25.—COLUMNS 20 FT. LONG, PLAIN CONCRETE.

take the point on the stress diagram where it changes from a straight line to a curve? A strict application of Johnson's rule leads to very low values in some cases, particularly for columns similar to Nos. 7, 8 and 9, because the stress-deformation diagrams are curved practically throughout their entire lengths. Moreover, the curves are frequently so flat that points of tangency

are not clearly defined. Nevertheless the approximate values of yield points plotted in Figs. 14 and 15 seem to be sufficiently accurate to warrant their use.

Attention is called to the various figures indicating the stress deforma-

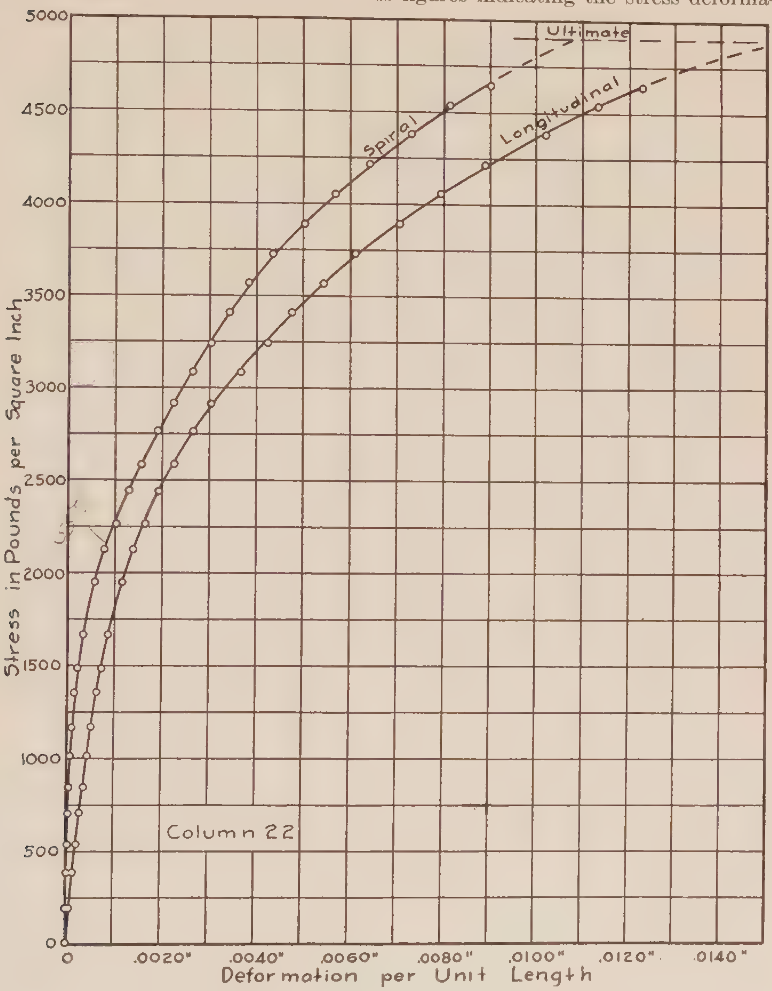

FIG. 26.—COLUMN 5 FT. LONG, 1 PER CENT SPIRAL.

tion curves for the individual columns and particularly to those which show repetition of loads.

It would be interesting to deduce formulas for the strength of reinforced concrete columns both with longitudinal and with spiral reinforcements but to do this properly would make the paper unduly long and it seems better to reserve the discussion of the theory for another paper.

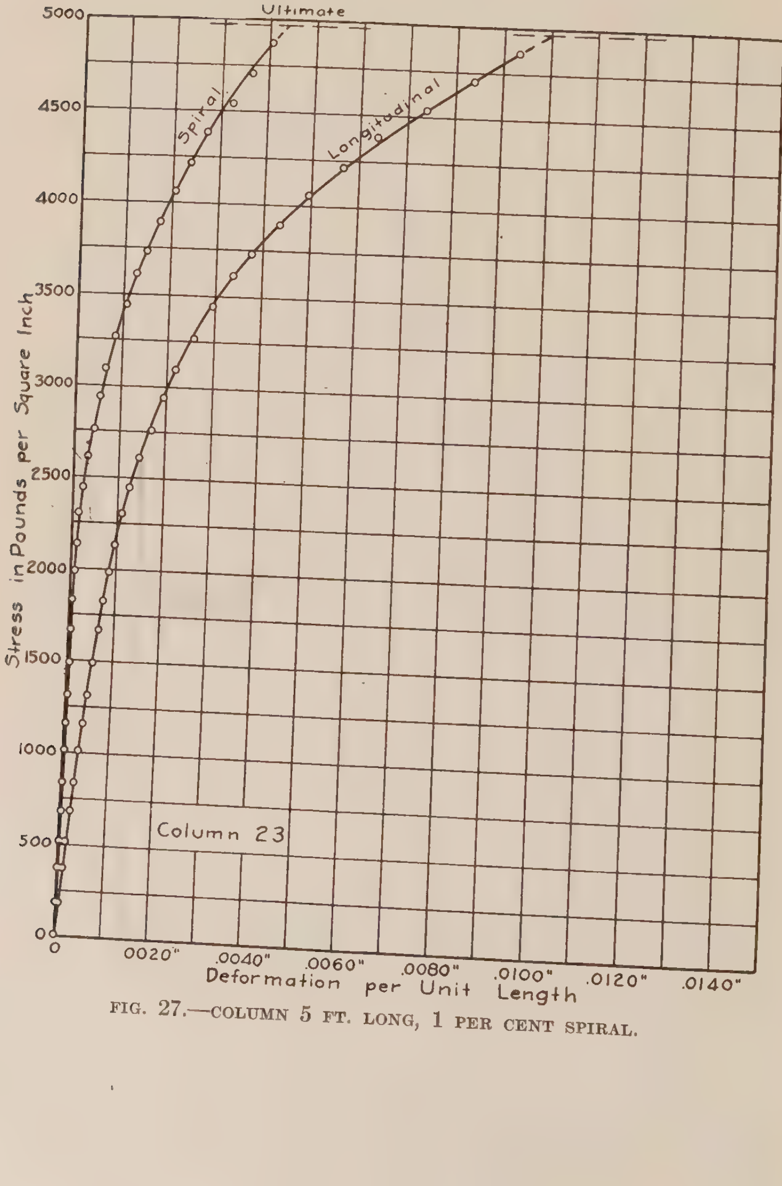

FIG. 27.—COLUMN 5 FT. LONG, 1 PER CENT SPIRAL.

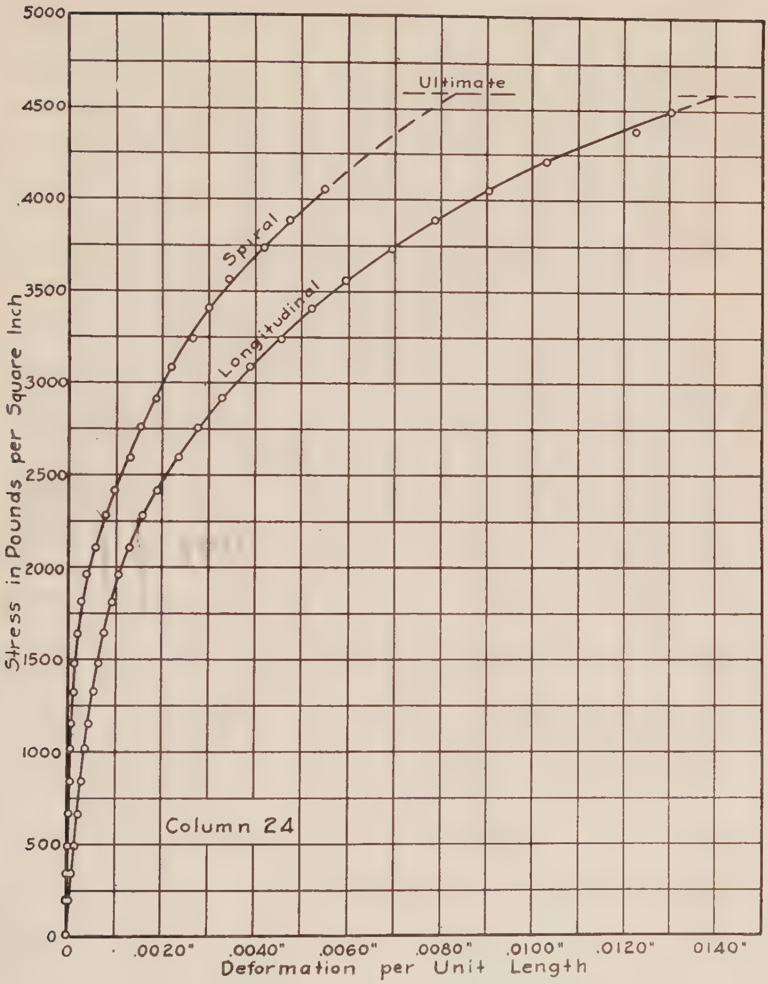

FIG. 28.—COLUMN 5 FT. LONG, 1 PER CENT SPIRAL.

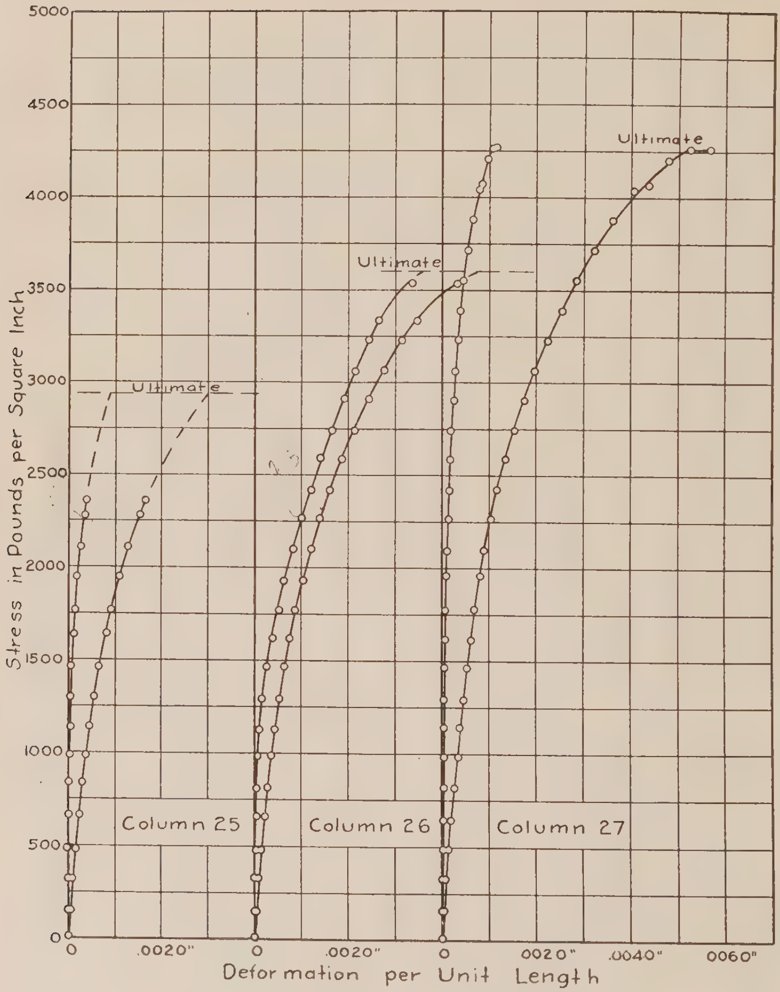

FIG. 29.—COLUMNS 20 FT. LONG, 1 PER CENT SPIRAL.

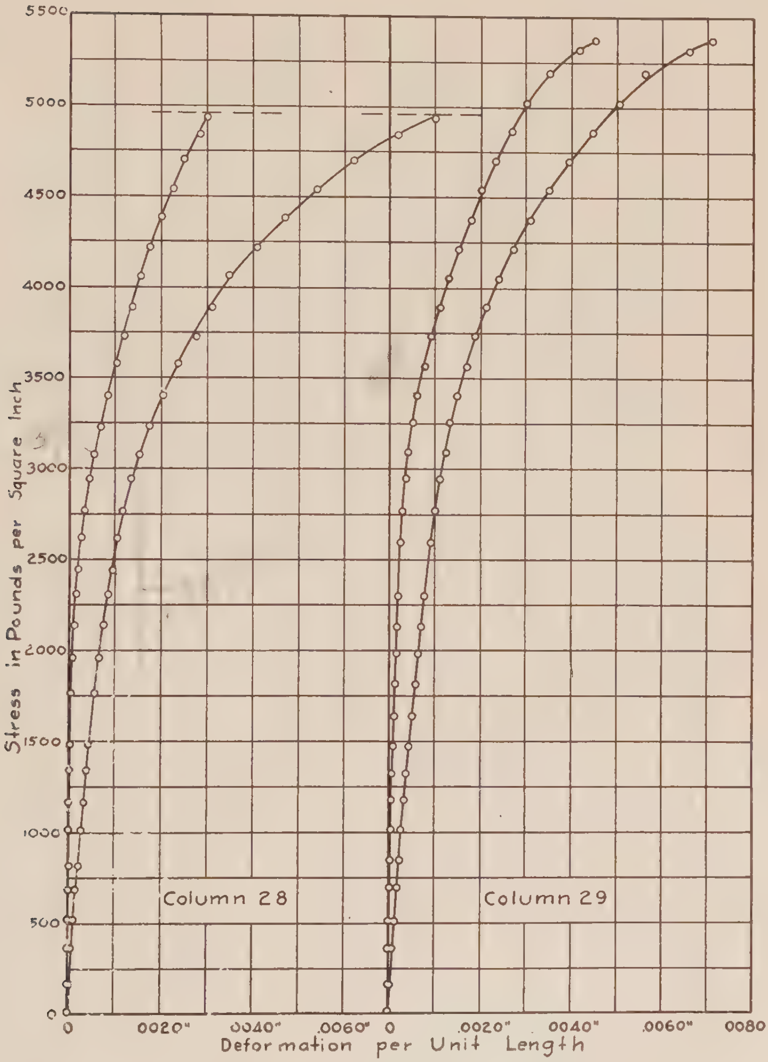

FIG. 30.—COLUMNS 10 FT. LONG, 1 PER CENT SPIRAL, 2 PER CENT VERTICAL STEEL.

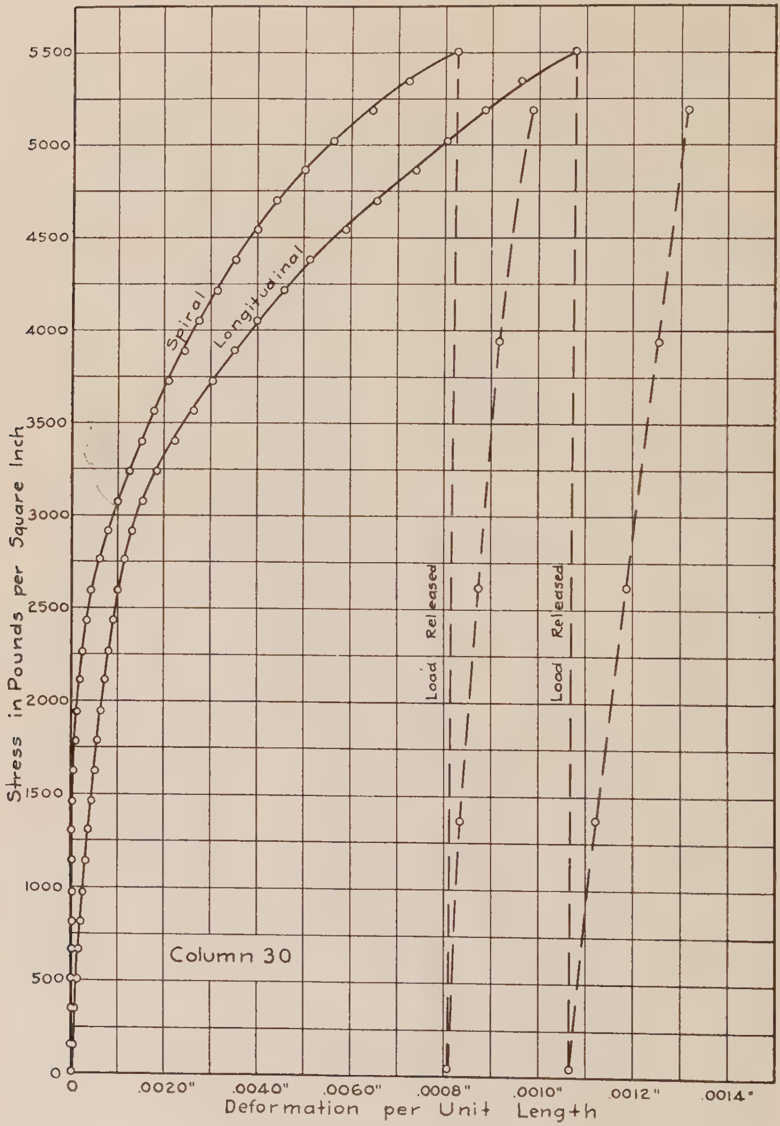

FIG. 31.—COLUMN 10 FT. LONG, 1 PER CENT SPIRAL, 2 PER CENT VERTICAL STEEL.

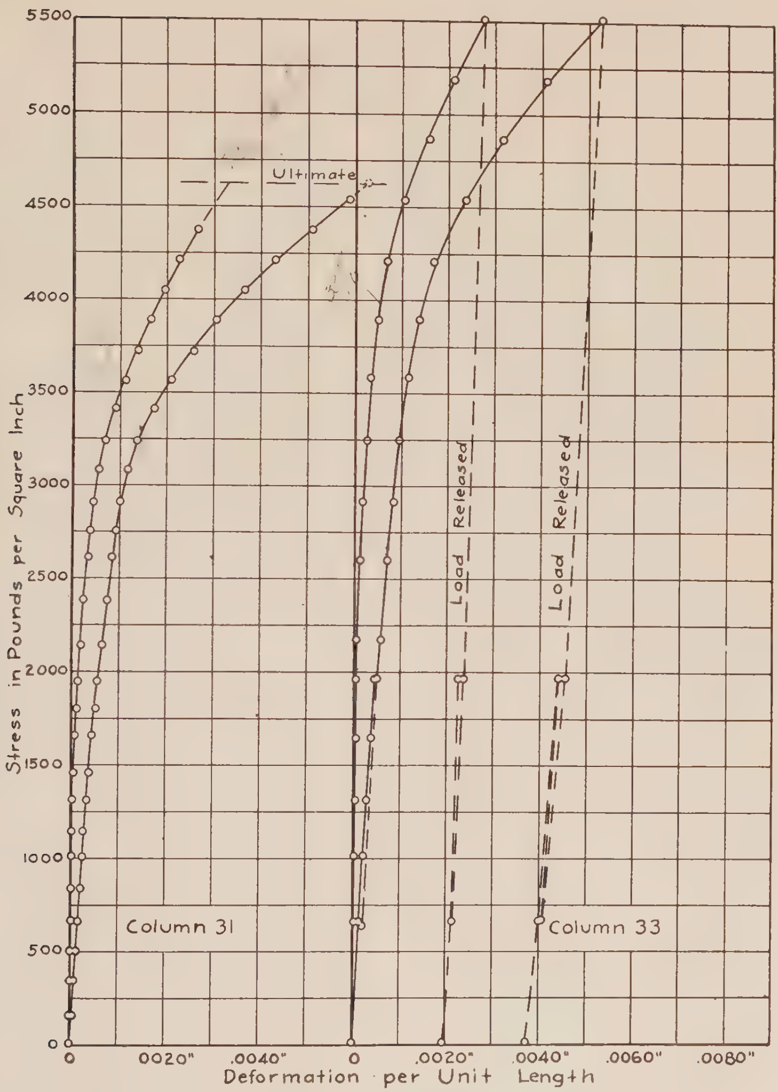

FIG. 32.—COLUMNS 10 FT. LONG, 1 PER CENT SPIRAL, 4 PER CENT VERTICAL STEEL.

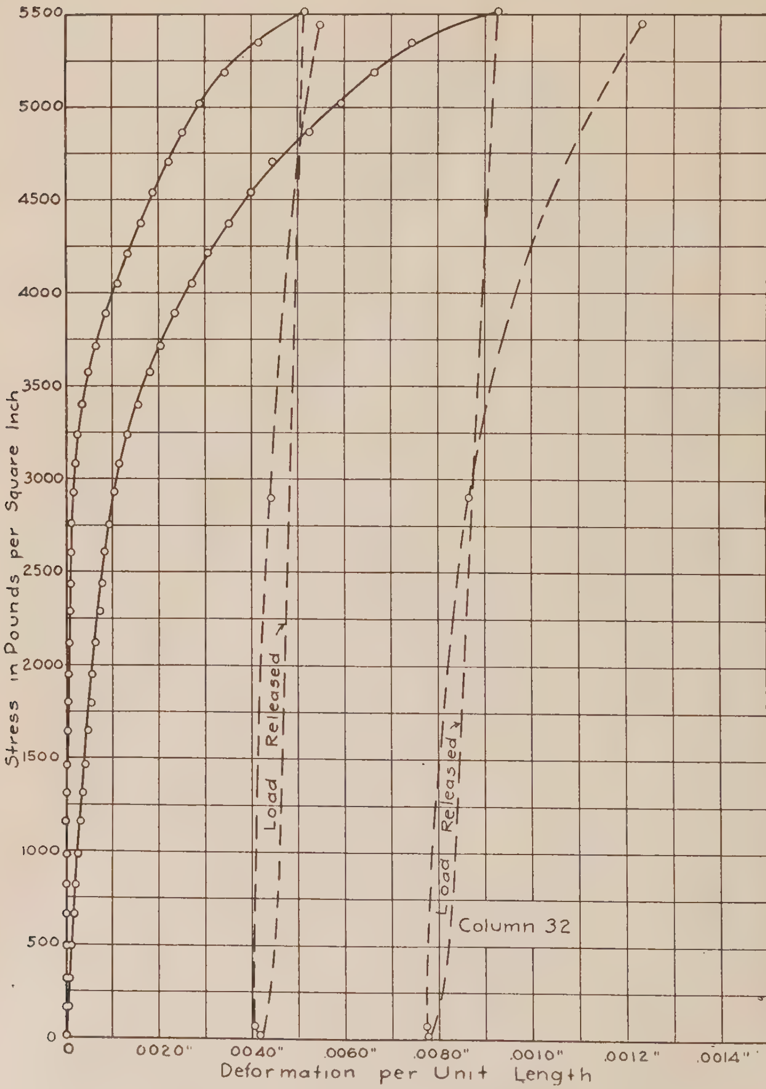

FIG. 33.—COLUMN 10 FT. LONG, 1 PER CENT SPIRAL, 4 PER CENT VERTICAL STEEL.

INFLUENCE OF TEMPERATURE ON THE STRENGTH OF CONCRETE.

By A. B. McDaniel.*

Concrete as a structural material is taking a foremost place in the field of construction, and has shown its adaptability to a wide range of uses and its reliability under various conditions of loading, climate, etc. The stress of competition and the demand for quick construction have necessitated the continuation of work regardless of climatic conditions or the season of the year. Until recent years it was the general practice to discontinue concrete work during cold weather.

Many failures of concrete structures have been caused by the placing of concrete in cold weather as in warm weather. One of the principal sources of failure is the removal of forms before the concrete has attained sufficient strength. It is often difficult, especially under cold weather conditions, to determine, even by close inspection, the actual strength of concrete. Hence it is desirable to have available, for the service of the engineer and the builder, information concerning the strength of concrete which has been subjected to varied temperature conditions.

It is the purpose of this paper to present a brief statement of the results of an investigation which the writer has made during the past two years at the University of Illinois and also of some tests made about four years ago during the construction of some buildings in Chicago. These tests were not sufficiently comprehensive to give conclusive results, but it is believed that they may furnish some suggestive information as to the strength of concrete at early ages under different temperature conditions.

I. Laboratory Tests.

Preliminary.—The laboratory tests were made in the Laboratory of Applied Mechanics of the University of Illinois. The work was done under the supervision of the author.

The tests of Groups I and II, 1913 Series, were made by J. Albert Anderson and W. J. Bublitz, senior civil engineering students of the class of 1914; and furnished the subject matter of their baccalaureate thesis. The tests of Group III, 1914 Series, were made by J. Albert Anderson, a graduate student in the Department of Civil Engineering; and special credit is due Mr. Anderson for the preparation of the tables and diagrams. All the tests were made with painstaking care and faithful attention to uniformity and accuracy of manipulation.

Concrete Materials.—The quality of the materials may be taken as representative of those used in first-class concrete work in the Middle West.

The cement was furnished by the Universal Portland Cement Company.

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Samples of the cement were taken at the beginning of each series of tests and were tested for fineness, soundness and tensile strength. The cement passed the requirements of the standard specifications of the American Society for Testing Materials.

The sand used came from a deposit of glacial drift near the Wabash River, at Attica, Ind. The material was clean and well graded.

The crushed limestone came from Kankakee, Ill. The material used in the first two groups of tests contained 87 per cent material of smaller than ½ in. and 46 per cent of material smaller than ¼ in. The stone used in Group III was well graded.

Concrete.—All the concrete was composed of 1 part cement, 2 parts sand, and 4 parts broken stone, by weight; or 1 part cement, 2.2 parts sand, and 3.6 parts broken stone, by volume. The materials for each specimen were weighed out separately and then thoroughly mixed by hand.

Molding and Storage of Test Specimens.—The specimens were classified according to the form of test specimen and storage conditions. Table I gives the details of the classification.

The Specimens of Group I of the 1913 Series were molded in the storage rooms under the following temperatures: Set A at 32° F., Set B at 65° F., and Set C at 84° F. The specimens of Group II of the 1913 Series were molded in the cement laboratory at the following temperatures: Set D at 77° F., Set E at 75° F., and Set F at 71° F. The specimens of Group III, 1914 Series, were molded in the concrete room of the Engineering Experiment Station at a temperature of 68° F. The specimens of Groups II and III were moved to their respective storage rooms after a set of six hours.

Table II shows the weight of the dry materials, the percentage of water in terms of the total dry materials, the temperature of the room and of the concrete, and the average time of molding.

The temperatures of the storage rooms were determined by daily readings of maximum and minimum thermometers. The temperatures for the several groups are given in Fig. 1 to 10.

All the specimens while in storage were covered with several layers of moist sacking, which was sprinkled daily.

Method of Testing.—All the specimens of Group I were taken from their storage places to the Laboratory of Applied Mechanics of the University of Illinois the day before they were tested. They were measured and weighed, their bearing surfaces coated with plaster of paris, and then were left in the open air of the laboratory for about 20 hours under a temperature of about 70° F.

The specimens of Group II were tested after about one hour from the time of their removal from the storage rooms. Two specimens of Set F, designated as F_{17} and F_{18}, after being stored under an average mean daily temperature of 27.1° F. for 44 days, were stored in the testing laboratory under an average mean daily temperature of 70° F., the former for 7 days and the latter for 21 days.

The specimens of Group III were brought to the testing laboratory from their storage places, weighed, measured, plastered, and tested within one

hour. The specimens of Set G, which were stored under freezing temperatures, were allowed to thaw out before being tested.

In the tests a spherical-seated bearing block was used.

Observed Results.—The results of the tests are given in Tables III to XI and in Figs. 1 to 10.

Standardized Strength.—Since a cube or a cylinder having a height equal to its diameter, tested for compressive strength, may be expected to give a

Table I.—Description of Test Specimens.

Series.	Group.	Set.	Specimens. Number.	Specimens. Form.	Number and Age of Specimens when Tested.
1913	I	A B C	15 15 15	6 x 6-in. cylinders	5 specimens of each set, at 7, 14, and 28 days.
1913	II	D E F	15 18 18	6-in. cubes	3 specimens of each set, at 4, 7, 11, 14, and 28 days.
1914	III	G H I M	15 15 15 15	8 x 16-in. cylinders	3 specimens of each set, at 3, 7, 10, 14, and 28 days.

Table II.—Data Concerning Molding of Specimens.

Type of Specimen.	Set.	Average Time of Molding, minutes.	Average Temperature, deg. F.		Weights of Materials, lb.			Water, per cent.*
			Air.	Concrete.	Cement.	Sand.	Stone.	
6-in. cylinders	A B C	8.5 8.5 8.5	32 65 84	70 71 70	2.17 2.17 2.17	4.34 4.34 4.34	8.68 8.68 8.68	10.0 10.0 10.0
6-in. cubes	D E F	7.0 7.0 7.0	77 75 71	70 70 69	2.42 2.42 2.42	4.84 4.84 4.84	9.68 9.68 9.68	10.0 11.0 10.0
8 x 16-in. cylinders	G H I M		68	69	10.2	20.4	40.8	9.3

* The concrete used in Groups I and II was of a medium or quaking consistency; while that used in Group III was wet, and was similar in consistency to that used in concrete building construction.

value which is higher than the representative compressive strength of the material, it seems desirable for the purposes of comparison to reduce the observed values for the cubes and short cylinders of Groups I and II to what may be considered as the equivalent values which would be obtained from cylinders of height equal to twice their diameter. To do this the values for the cubes and cylinders have been multiplied by 0.73, which is the ratio of strength of prisms to strength of cubes determined by the Committee on

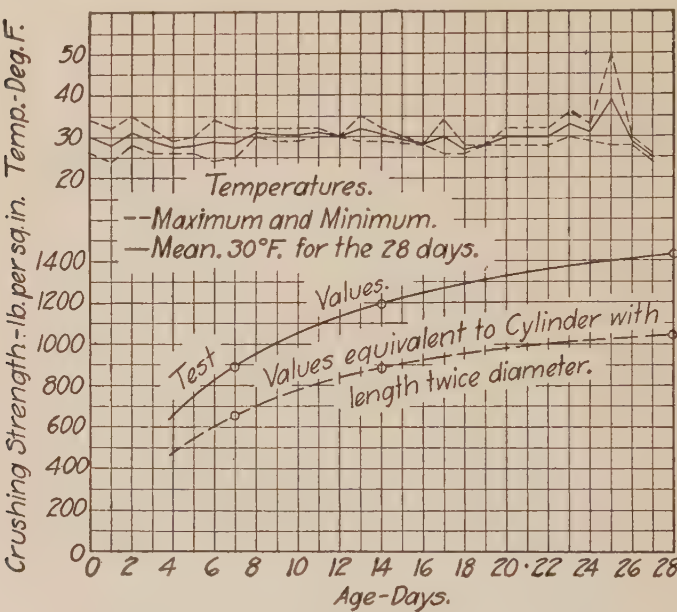

FIG. 1.—SET A, GROUP I, 1913 SERIES.

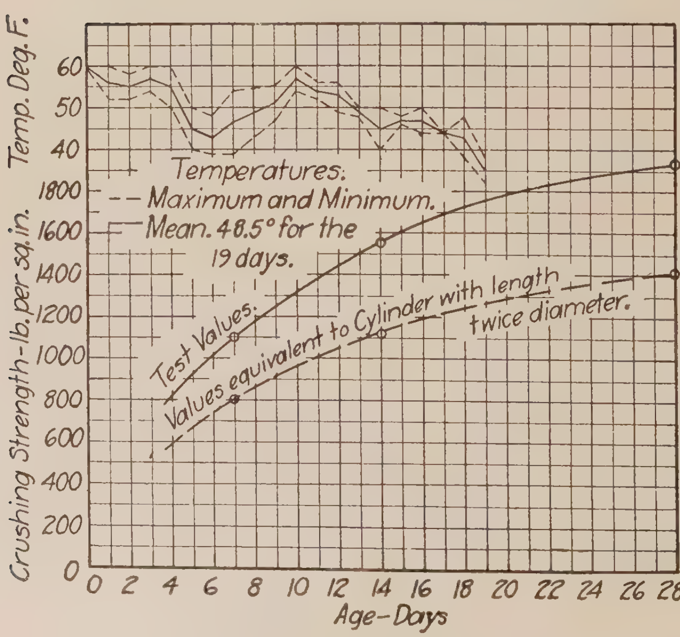

FIG. 2.—SET B, GROUP I, 1913 SERIES.

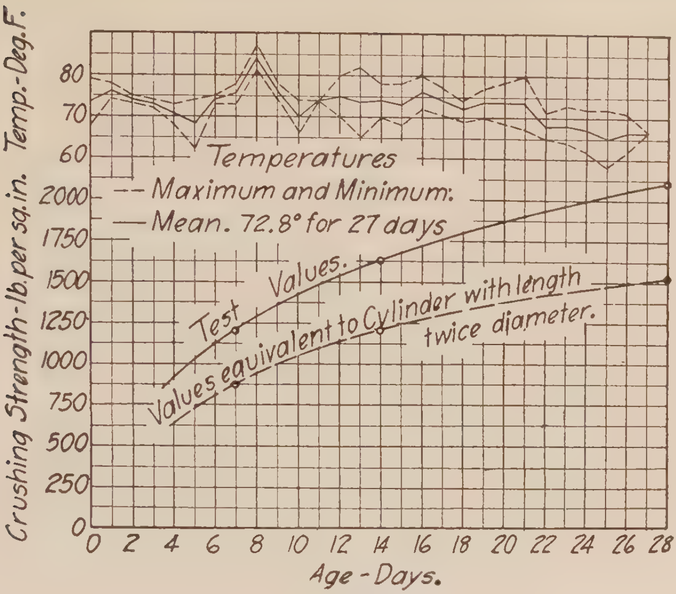

FIG. 3.—SET C, GROUP I, 1913 SERIES.

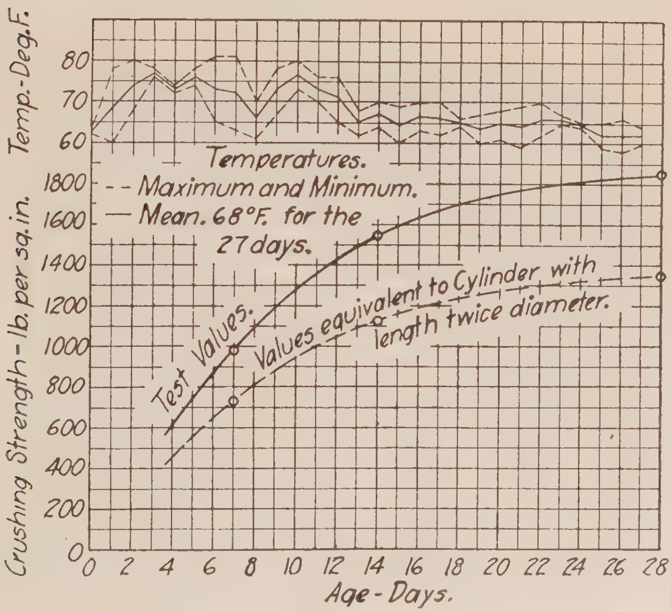

FIG. 4.—SET D, GROUP II, 1913 SERIES.

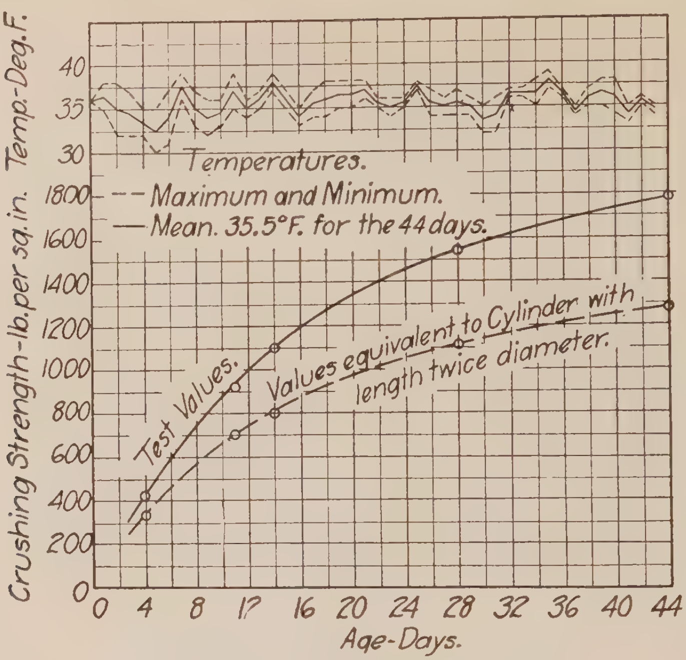

FIG. 5.—SET E, GROUP II, 1913 SERIES.

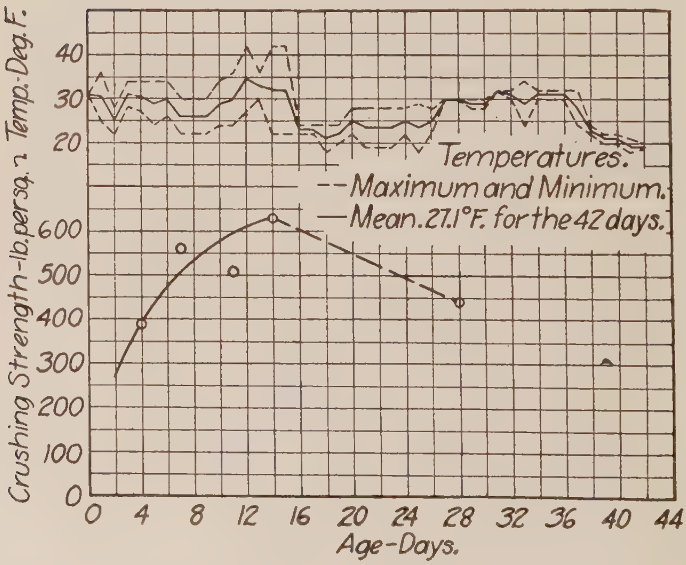

FIG. 6.—SET F, GROUP II, 1913 SERIES.

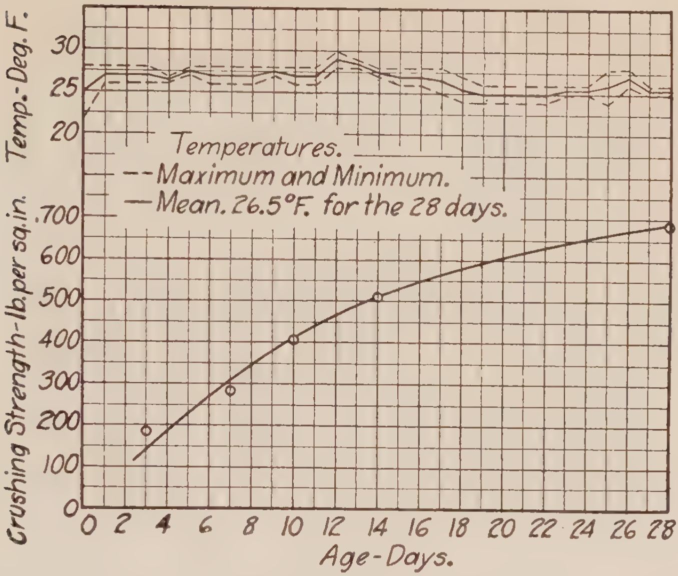

FIG. 7.—SET G, GROUP III, 1914 SERIES.

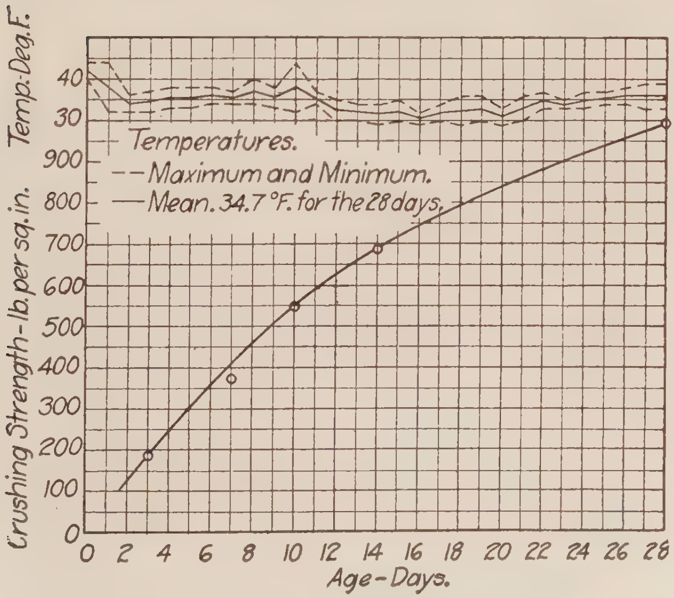

FIG. 8.—SET H, GROUP III, 1914 SERIES.

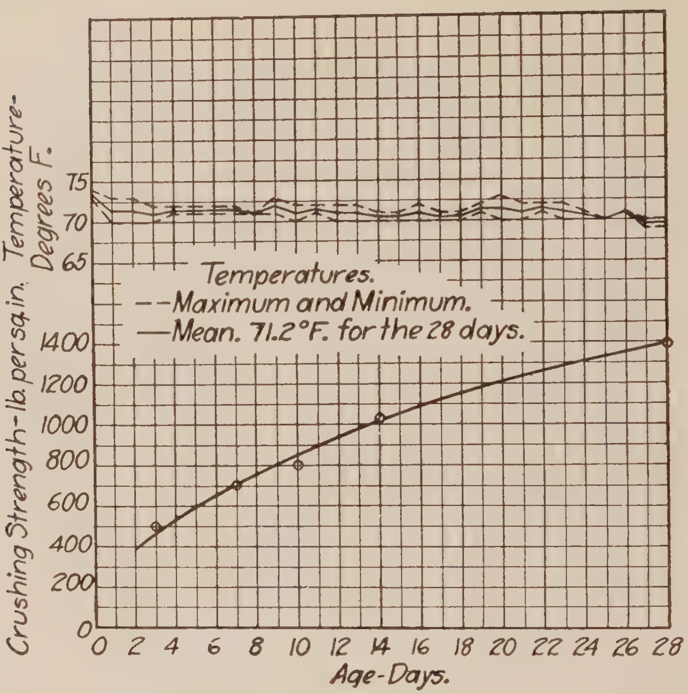

FIG. 9.—SET I, GROUP III, 1914 SERIES.

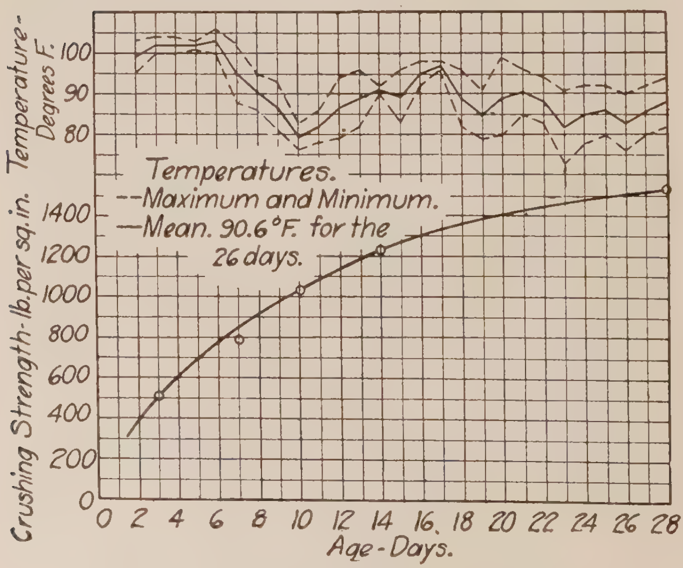

FIG. 10.—SET M, GROUP III, 1914 SERIES.

Specifications and Methods of Tests for Concrete Materials of the American Concrete Institute. The reduced values are designated as the standardized strengths in Tables III to XI, and are shown by the lower curve in Figs. 1 to 10.

Group I.—The results of the tests of Group I, 6 by 6-in. cylinders, are given in Tables III, IV, and V; and the relation between strength and age is shown in Figs. 1, 2, and 3. The curves are drawn through the average values for each group of five specimens, for 7, 14 and 28 days. At the top of each figure is shown the temperature conditions for that set; the maximum, the minimum, and the mean temperatures.

Group II.—The results of the tests of Group II, 6-in. cubes, are given in Tables VI to XI; and the relation between strength and age is shown in Figs. 4, 5 and 6. The strength and temperature curves are drawn as stated for Group I.

The Sets D and E, Figs. 4 and 5, were stored under substantially uniform temperature conditions, and give results of practically the same character as those of Group I.

The specimens of Set F were stored in a room where it was known the temperature would not be uniform. All of the specimens tested at 11 days were slightly disintegrated on the surface, and those tested at 28 days were badly disintegrated; while of those reserved to be tested at 42 days only one could be tested at that date, the remaining specimens, F_{17} and F_{18}, being very badly disintegrated. Specimen F_{17} was tested at 49 days, and F_{18} at 63 days. Since there was only one specimen at each of these ages, and since none of the other groups contained specimens at corresponding ages, the results of these two tests are not plotted in Fig. 6, and are not further considered.

The results of Set F indicate that the low temperature retarded the hardening action of the concrete, and that the alternations above and below freezing caused a softening and crumbling of the material.

Group III.—The results of the tests of Group III, 8 by 16-in. cylinders, are given in Tables XII to XVI; and the relation between strength and age is shown graphically in Figs. 7 to 10. It is noteworthy that under a temperature slightly below freezing the concrete gained strength continuously. (See Fig. 7.) It is also interesting to note that the curve for a mean temperature of 26.5° F. is substantially of the same character as that for a mean temperature of 71.2° F. (Compare Fig. 7 and Fig. 9.)

Summary.—The results for the three sets of Group I are presented in Fig. 11, and the corresponding values for Groups II and III are given in Figs. 12 and 13. Figs. 11 to 13 show the relation between strength and age for the several mean temperatures.

In Group I the test specimens were cylinders 6 in. in diameter and 6 in. high, and in Group II the specimens were 6-in. cubes; and owing to the effect of the restraint of the pressing surfaces of the testing machine, the results of these tests are not further considered.

In Group III the test specimens were cylinders 8 in. in diameter and 16 in. high, and the interpolated results for these tests are presented in Fig. 14 to show the relation between strength and temperature for the several ages.

Table III.—Compressive Strength—Age 7 Days.

Group I. 6 x 6-in. Cylinders.

Set.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Standardized Strength, lb. per sq. in.	Remarks.
	6.0	29 240	1030			
	6.18	27 670	930			
A	6.06	24 890	870	890	650	Cracked uniformly around circumferential area.
	6.06	22 450	780			
	6.06	24 450	850			
	6.12	36 130	1230			Cracked uniformly.
	6.00	34 770	1230			" "
B	6.00	31 660	1120	1100	800	" "
	5.87	25 030	930			Skewed.
	5.87	26 950	980			"
	5.87	27 080	1000			
	5.81	35 830	1350			
C	5.94	37 660	1370	1200	880	Visible voids.
	5.87	31 140	1150			
	6.00	32 250	1140			

Table IV.—Compressive Strength—Age 14 Days.

Group I. 6 x 6-in. Cylinders.

Set.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Standardized Strength, lb. per sq. in.	Remarks.
	6.0	38 020	1340			
	6.12	33 340	1140			
A	6.0	39 220	1390	1190	870	Cracked uniformly around circumferential area.
	6.0	31 390	1110			
	5.87	26 700	980			
	6.0	47 090	1670			Cracked uniformly.
	5.94	50 460	1820			" "
B	5.97	45 850	1640	1540	1130	" "
	5.94	30 000	1090			Badly skewed.
	5.87	40 640	1500			Slightly skewed.
	5.87	46 170	1700			
	6.0	50 000	1770			
C	6.12	44 190	1510	1660	1210	Uniform throughout.
	6.0	44 420	1570			
	5.87	47 100	1740			

Table V.—Compressive Strength—Age 28 Days.

Group I. 6 x 6-in. Cylinders.

Set.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Standardized Strength, lb. per sq. in.	Remarks.
A	6.06	39 340	1370	1430	1040	All uniform.
	6.06	37 730	1320			
	5.94	48 450	1750			
	6.12	37 660	1280			
	6.00	40 300	1430			
B	6.06	56 240	1960	1940	1410	Area reduced by visible voids.
	6.0	55 300	1950			
	6.06	54 600	1900			
	6.0	34 670	1230			
	6.0	40 000	1420			
C	5.97	55 720	2000	2090	1530	
	5.94	63 650	2310			
	5.87	60 260	2220			
	5.87	49 760	1840			Slightly skewed.
	6.0	40 390	*1430			Badly skewed.

* Not used in calculating average.

Table VI.—Compressive Strength—Age 4 Days.

Group II. 6-in. Cubes.

Set.	Weight, lb.	Size, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Standardized Strength, lb. per sq. in.	Remarks.
D	18.50	6 x 6 x 6	28 400	790	780	570	
	18.50	"	22 450	620			
	18.50	"	33 340	920			
E	19.00	"	16 690	460	450	330	
	19.00	"	10 000	280			
	18.75	"	21 300	590			
F	18.75	"	15 680	440	390	280	Slight coating of frost, but all had uniform break.
	18.75	"	13 050	360			
	18.75	"	13 000	360			

Table VII.—Compressive Strength—Age 7 Days.

Group II. 6-in. Cubes.

Set.	Weight, lb.	Size, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Standardized Strength, lb. per sq. in.	Remarks.
D	18.75	6 x 6 x 6	39 390	1090	980	720	
	19.00	"	35 930	1000			
	18.75	"	31 300	860			
E		"	17 100	470	470	340	Broke uniformly.
		"	19 820	550			
		"	14 060	390			
F	18.50	"	20 880	580	560	410	
	18.75	"	19 230	530			
	18.75	"	20 760	580			

TABLE VIII.—COMPRESSIVE STRENGTH—AGE 11 DAYS.

Group II. 6-in. Cubes.

Set.	Weight, lb.	Size, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Standardized Strength, lb. per sq. in.	Remarks.
D	18.75	6 x 6 x 6	43 420	1200	1320	970	
	18.75	"	52 440	1460			
	18.75	"	47 300	1300			
E	19.00	"	42 310	1180	920	670	Visible voids.
	19.00	"	31 060	860			Broke at one corner.
	18.75	"	26 000	720			
F	18.75	"	15 080	420	500	370	
	18.75	"	17 760	490			
	18.75	"	21 820	610			

TABLE IX.—COMPRESSIVE STRENGTH—AGE 14 DAYS.

Group II. 6-in. Cubes.

Set.	Weight, lb.	Size, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Standardized Strength, lb. per sq. in.	Remarks.
D	19.00	6 x 6 x 6	58 710	1630	1540	1130	
	19.00	"	62 810	1740			
	18.75	"	45 530	1260			
E		"	38 630	1070	1100	800	
		"	40 300	1120			
		"	40 280	1120			
F	18.75	"	19 330	540	640	470	
	18.75	"	26 350	730			
	18.75	"	22 900	650			

TABLE X.—COMPRESSIVE STRENGTH—AGE 28 DAYS.

Group II. 6-in. Cubes.

Set.	Weight, lb.	Size, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Standardized Strength, lb. per sq. in.	Remarks.
D	19.00	6 x 6 x 6	64 000	1780	1850	1350	
	19.00	"	67 460	1870			
	19.00	"	68 440	1900			
E	18.75	"	52 590	1460	1550	1130	
	18.75	"	58 270	1620			
	18.75	"	56 350	1560			
F	19.00	"	16 540	460	440	320	
	19.00	"	15 160	420			
	18.75	"	10 400	*290			

* Specimen in bad condition; not included in average.

Table XI.—Compressive Strength—Age 42 Days.

Group II. 6-in. Cubes.

Set.	Weight, lb.	Size, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Standardized Strength, lb. per sq. in.	Remarks.
E	18.75 18.75 18.75	6 x 6 x 6 " "	66 710 62 880 62 240	1850 1740 1740	1780 1780	1300 1300	Broke uniformly. One corner broke. Slightly skewed.
F	18.00	" 5 x 5 x 6 4 x 5 x 6	15 240 31 720 8 040	420 1270* 400†			Specimens in a soft and crumbling condition.

* Age when tested, 49 days.
† Age when tested, 63 days.

Table XII.—Compressive Strength—Age 3 Days.

Group III, 1914 Series. 8 x 16-in. Cylinders.

Set.	Weight, lb.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Remarks.
G	67.75 67.25 70.25	7.94 7.87 8.06	8 880 9 720 9 340	180 200 180	190	Crumbled. Crumbled badly.
H	65.0 66.0 65.0	7.94 7.87 8.0	9 950 8 250 9 250	200 170 180	180	 Plaster loose on one end.
I	69.0 70.25 69.75	8.0 8.06 8.06	29 650 23 750 22 600	600 460 440	500	
M	64.0 65.0 65.0	8.0 7.94 7.94	24 000 30 850 20 000	480 620 400	500	

Table XIII.—Compressive Strength—Age 7 Days.

Group III, 1914 Series. 8 x 16-in. Cylinders.

Set.	Weight, lb.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Remarks.
G	67.75 67.25 65.50	8.0 8.0 8.0	17 200 14 750 10 700	340 290 210	280	
H	66.0 69.5 65.5	7.87 8.06 7.94	14 450 18 920 18 530	300* 370 370	370	Skewed one inch, horizontal crack.
I	67.75 67.25	8.0 7.94 8.12	40 800 31 630 34 340	810 640 660	700	
M	68.0 69.0 69.0	8.0 8.06 8.06	44 500 40 250 35 150	890 790 690	790	

* Not used in calculating average strength.

Table XIV.—Compressive Strength—Age 10 Days.

Group III, 1914 Series. 8 x 16-in. Cylinders.

Set.	Weight, lb.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Remarks.
G	66.5 67.5 70.5	8.0 7.87 8.0	18 670 18 280 23 630	370 370 470	400	
H	68.0 68.5 69.0	8.0 8.12 8.06	25 000 30 680 26 830	500 600 530	540	Two 8 x 8 forms.
I	67.5 68.0	8.06 8.0	44 700 36 050	880 720	800	Top crumbled. Skewed. Fractured in transit.
M	68.0 69.0 69.5	7.94 8.06 8.06	59 620 46 700 49 540	1200 920 970	1030	 Two 8 x 8 forms.

Table XV.—Compressive Strength—Age 14 Days.

Group III, 1914 Series. 8 x 16-in. Cylinders.

Set.	Weight, lb.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Remarks.
G	70.0 68.5 69.5	8.0 7.94 8.06	26 430 25 330 25 420	520 510 500	510	
H	67.5 64.5 66.5	7.94 8.06 8.0	30 550 35 100 37 750	620 690 750	690	
I	66.0 69.5 67.0	7.94 8.0 8.0	51 000 63 230 41 430	1030 1260 820	1040	Skewed slightly. Bearing faces not parallel.
M	67.5 66.5 66.5	8.0 7.94 8.0	58 000 40 650 65 000	1150 820* 1290	1220	Visible voids.

* Not used in calculating average strength.

Table XVI.—Compressive Strength—Age 28 Days.

Group III, 1914 Series. 8 x 16-in. Cylinders.

Set.	Weight, lb.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Remarks.
G	68.0 67.5 65.0	8.0 7.94 7.94	32 100 35 900 34 100	630 730 690	680	
H	65.0 65.0 64.0	8.06 8.0 7.87	45 900 51 600 51 400	900 1030 1050	990	
I	68.5 68.5 66.0	8.0 8.0 7.87	83 950 60 000 63 900	1670 1190 1290	1380	Odd fracture.
M	66.0 66.0 66.0	8.0 8.12 8.0	63 500 101 900 68 200	1260 1960 1360	1530	

Fig. 14 may be employed to determine (1) the strength which the concrete attained at different ages under a constant temperature, (2) the age at which a particular strength was gained under the different temperatures, and (3) the strength which may be expected at different ages under different tem-

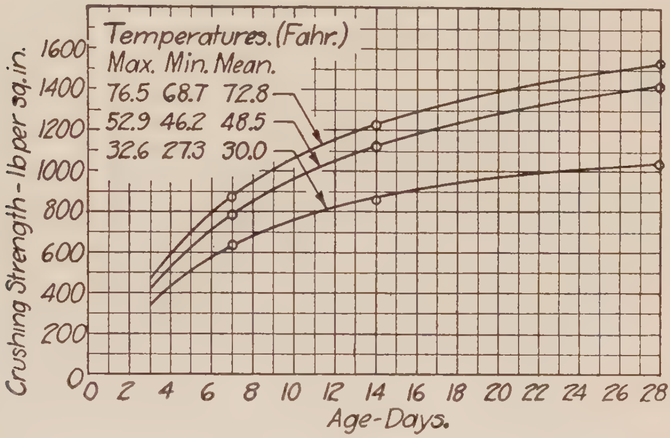

FIG. 11.—GROUP I, 1913 SERIES. STANDARDIZED VALUES.

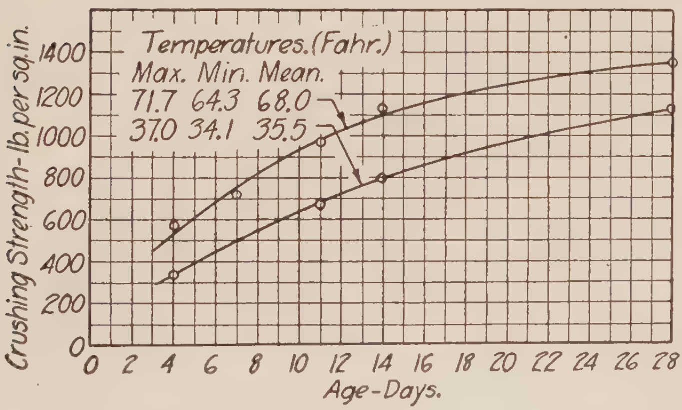

FIG. 12.—GROUP II, 1913 SERIES. STANDARDIZED VALUES.

peratures. The relative strength attained by concrete at different temperatures during the hardening and at different ages may be expected to vary somewhat with differences in cements, aggregates, and consistencies; but it is thought that the values in Fig. 14 may be taken to represent the effect of the variation in the temperatures during hardening upon the strength.

Fig. 15 has been drawn by taking values from the curves in Fig. 14. It shows, in a general way, the relation between the strength at 28 days under 70° F. and strength attained at various ages under varying temperatures. Fig. 15 can be used in substantially the same way as Fig. 14.

The tests summarized in Figs. 14 and 15 cover a wide range of temperature conditions, the average temperature varying from 20.4° to 90.6° F., and are fairly consistent; and hence it is believed these values are sufficiently accurate to furnish suggestive information which may be useful in determining the time when forms may be removed and loads applied.

II. Chicago Building Tests.

General Description.—All the tests were made under the supervision of Mr. W. A. Hoyt, Consulting Engineer of Chicago, to whom the author is indebted for the data given in the following statement:

The tests of Group I, 1910–11 Series, were carried on in January, February and March, 1911, during the construction of a reinforced concrete warehouse

Table XVII.—Data Concerning Materials and Mixing of Concrete.

Group.	Proportions of Materials by Volume.	Heating of Materials.	Method of Mixing and Securing Concrete for Specimens.
I. 1910–11 Series	1:2:3½	Sand and water heated by steam pipe.	Hand mixing. Concrete taken from wheelbarrows.
II. 1910–11 Series	1:2:4	Sand, gravel and water heated by steam.	Cube mixer used. Concrete taken from wheelbarrows.
III. 1910–11 Series	1:3:4½	Sand and gravel heated over pipe. Water heated by steam.	Batch mixer used. Concrete taken from wheelbarrows.
IV. 1911–12 Series	1:2:4	Materials not heated. Temperatures were: sand 44° F., gravel 38° F., and water 39° F. Temperature of concrete was 40° F.	Batch mixer used. Concrete taken from mixer.

for the Garfield Park Storage Company. The building is 50 by 63 ft. and five stories in height. The walls are of brick, and the floors and roof are of the reinforced concrete girder and arched floor construction built in accordance with a system devised by the Henry Ericsson Company, who were the contractors for the building.

The tests of Group II, 1910–11 Series, were made during the construction of the Oak Park Warehouse in January, February and March, 1911. The building is 75 by 100 ft. and five stories in height. The walls are of brick and the floors and roof are of the slab and shallow girder type of reinforced construction, devised by Condron and Sinoks.

The tests of Group III, 1910–11 Series, were made at the Keelin Warehouse & Van Company during January, February and March, 1911. The building is 50 by 80 ft. and five stories in height. The walls are of brick and the floors and roof are of the reinforced concrete girder and arched floor construction built in accordance with a system devised by the Henry Ericsson Company, who were the contractors for the building.

The tests of Group IV, 1911–12 Series, were made during the construction of the Stewart building in the winter of 1911–12.

Concrete Materials.—The quality of the materials may be considered as representative of those used in first-class concrete construction in Chicago.

The cement was a standard brand of Portland cement. Tests for soundness, fineness, and tensile strength met the requirements of the Standard Specifications of the American Society for Testing Materials.

The sand used was a good quality of torpedo sand.

The gravel was a well washed and screened material.

Concrete.—The detailed information concerning the proportions of the dry materials by volume, the heating of the materials and the method of mixing and handling the concrete is given in Table XVII.

Molding and Storage of Specimens.—The specimens were molded and stored in the buildings under construction. Table XVIII gives the details of their classification.

Table XVIII.—Description of Test Specimens.

Series.	Group.	Set.	Specimens.		Number and Age of Specimens when Tested.
			Number.	Form.	
1910–11	I	R	15	8 x 16 in. cylinders	3 specimens at 8, 14, 21, 29 and 42 days.
		S	12	"	3 specimens at 8, 14, 36 and 43 days.
	II	T	15	"	3 specimens at 10, 14, 22, 30 and 43 days.
		V	15	"	3 specimens at 7, 15, 21, 29 and 43 days.
	III	W	15	"	3 specimens at 8, 14, 21, 28 and 41 days.
1911–12	IV	X	9	"	3 specimens at 7, 14 and 28 days.
		Y	9	"	3 specimens at 7, 14 and 28 days.
		Z	7	"	3 specimens at 7 and 14 days. 1 specimen at 28 days.

The specimens of Group I, 1910–11 Series, were stored on the mezzanine floor of the building. Protection was furnished below the floor by a canvas curtain and above the floor by a wooden housing. Salamanders were kept burning during the pouring of the concrete and for several days thereafter, upon the first and mezzanine floors during the pouring of the mezzanine and second floors, respectively. The specimens were covered with canvas.

The specimens of Set T, Group II, 1910–11 Series, were stored on the second floor of the building. The specimens of Set V were stored on the third floor for one night and the next day were removed in a frozen condition to a platform about 3 ft. above the second floor. During the concreting, salamanders were placed on the floor below the one being poured and were kept burning during the progress of the work and for four or five days after the floor was completed. The specimens were covered with tar paper.

The specimens of Group III, 1910–11 Series, were stored on a platform about 3 ft. above the third floor of the building. During the concreting,

salamanders were placed on the floor below and were kept burning during the progress of the work, and for seven or eight days after the floor was completed. The specimens were covered with tar paper.

The specimens of Set V of Group IV, 1911–12 Series, were stored for two days in the building and were then removed to Lewis Institute, where they were stored for 26 days. The specimens of Set Y were stored at the building near salamanders which were kept burning at intervals for two weeks. The

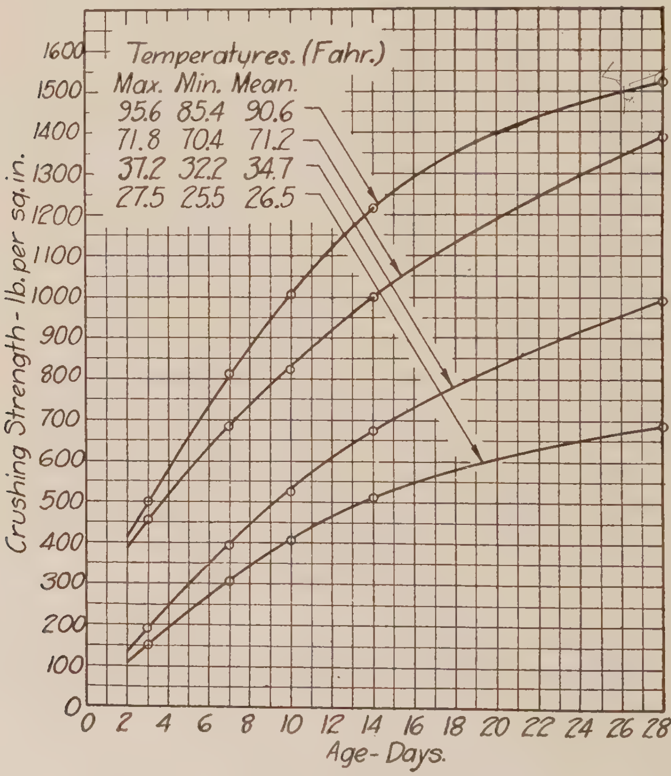

FIG. 13.—GROUP III, 1914 SERIES.

specimens of Set Z were stored in the building for 27 days in an unprotected location.

The temperature of the air adjacent to each set of specimens was determined from the occasional reading of a maximum and minimum thermometer. The temperature for the several groups are given in Figs. 16 to 23 inclusive.

Method of Testing.—The specimens were removed from their respective storage places to Lewis Institute where they were stored under a normal

temperature for 18 to 20 hours before testing. A standard make of testing machine was used.

Observed Results.—The results of the tests are given in Tables XXIV to XXXVII, and in Figs. 16 to 23.

Group I, 1910–11 Series.—The results of the tests of Group I are given in Tables XXIV, XXV, XXVI and XXVII, and the relation between strength and age is shown in Figs. 16 and 17. The curves are drawn through the average values for each group of three specimens for 8, 14, 21, 29 and 42

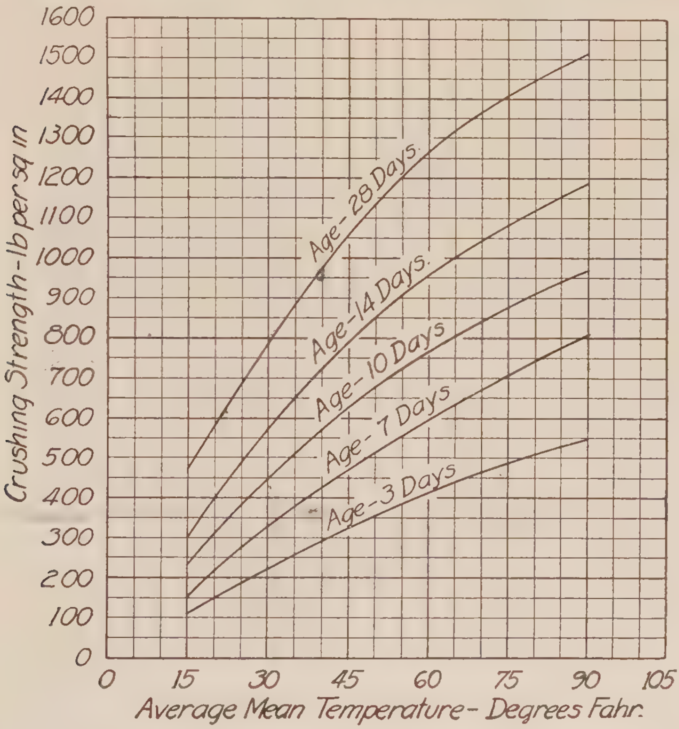

FIG. 14.—RELATION OF STRENGTH TO TEMPERATURE FOR DIFFERENT AGES.

days for Set R and for 8, 14, 36 and 43 days for Set S. At the top of each figure is shown the temperature conditions for that set; the maximum, the minimum, and the mean temperatures.

The results of Group I show the high rate of increase of strength gained by normal temperature conditions at early ages.

Group II, 1910–11 Series.—The results of the tests of Group II are given in Tables XXVIII, XXIX, XXX and XXXI, and the relation between strength and age is shown in Figs. 18 and 19. The strength and temperature curves are drawn similarly to those of Group I.

The results of Sets T and V show the difference in the rate of increase in strength at early ages under low and normal temperature conditions, respectively.

Group III, 1910–11 Series.—The results of the tests of Group III are given in Tables XXXII and XXXIII, and the relation between strength and age is shown graphically in Fig. 20. It is worthy of notice that under a temperature between 50° and 60° F. during the first seven days, there was a high rate of increase of strength, and that during the following 32 days under a

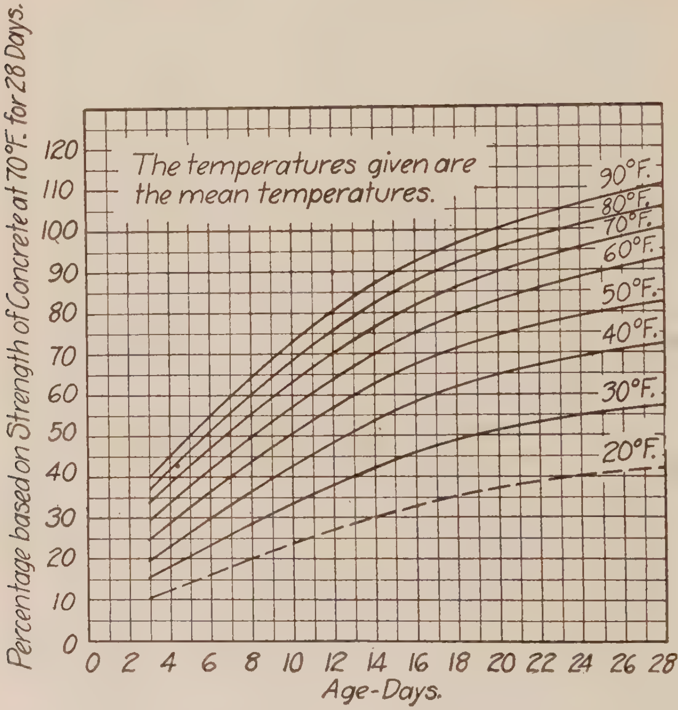

FIG. 15.—PERCENTAGES OF STRENGTH FOR DIFFERENT TEMPERATURES.

temperature of from 31° to 40° F. there was a lower but still a marked rate of increase in strength.

Group IV, 1911–12 Series.—The results of the tests of Group IV are given in Tables XXXIV, XXXV, XXXVI and XXXVII, and the relation between strength and age is shown graphically in Figs. 21, 22 and 23.

The results of Set X show the steady increase in strength which may be expected of concrete under a normal temperature of from 60° to 70° F. In Set Y, there was a uniform increase of strength for 13 days under an average temperature of about 44° F., followed by a reduction of strength

accompanied by two alternations of temperature above and below freezing. Similarly in Set Z, a gradual increase of strength for 14 days accompanied an average temperature of about 36° F., followed by a reduction of strength with alternations of temperature above and below freezing.

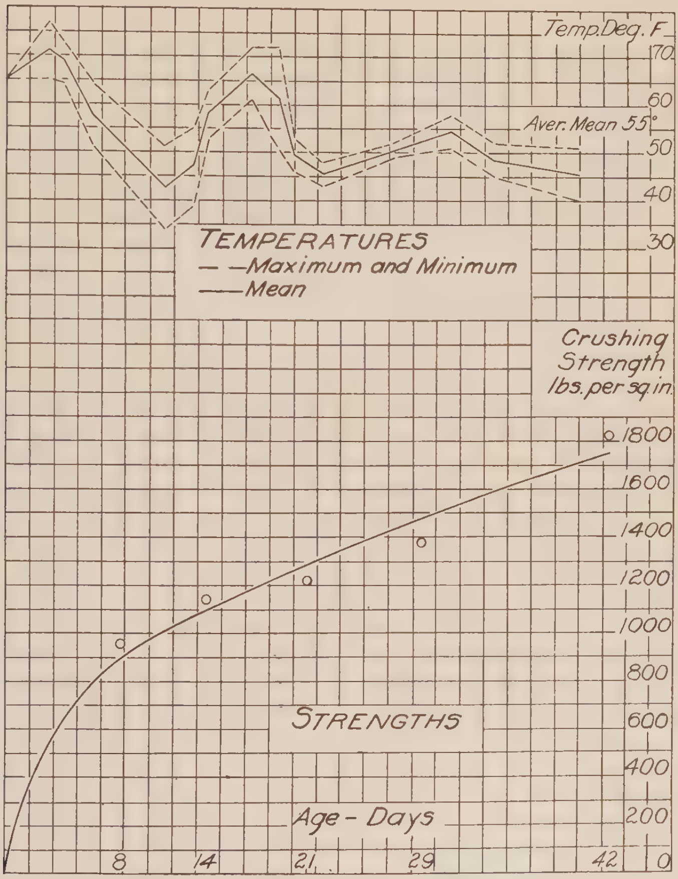

FIG. 16.—SET R, GROUP I, 1910–11 SERIES.

III. Discussion of Tests.

Preliminary.—The results of the tests given in Part II, are principally of interest and value as a measure of comparison with the results of the laboratory tests which are stated in Part I. A lack of specific and accurate information concerning the Chicago tests makes a close and detailed comparison an

impossibility, but the data available are sufficiently comprehensive to justify a study of the results, independently and in relation to those of the laboratory tests. It is to be noted, also, that different kinds of aggregates are used in the tests; Wabash building sand and crushed limestone in the laboratory tests and torpedo sand and gravel in the building tests. The proportions by

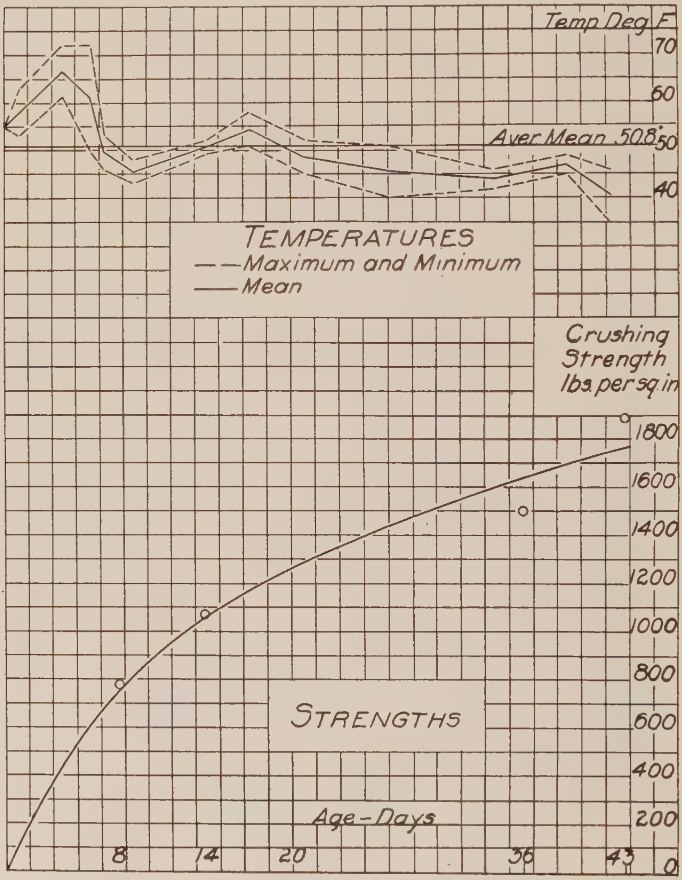

FIG. 17.—SET S, GROUP I, 1910–11 SERIES.

volume used in the building tests were slightly different from those used in the laboratory tests. These differences of materials and proportions may account to some extent for the higher results obtained in the building tests, especially those of Group I of the 1910–11 Series, where a $1:2\frac{1}{2}:3$ mixture was used.

Relation of Strength and Age.—An inspection of Figs. 7 to 23 inclusive shows that there is an increase of strength with age under fairly uniform temperature conditions. Both sets of tests show that the higher the temperature, the greater the rate of increase of strength during the same periods of time. However, a direct comparison between the increase of strength with

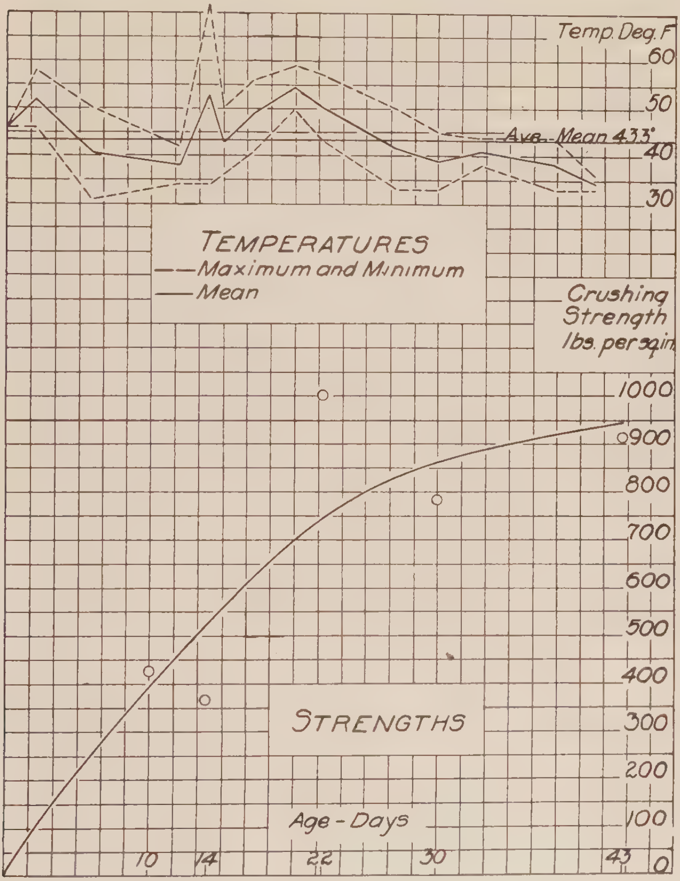

FIG. 18.—SET T, GROUP II, 1910–11 SERIES.

age for any temperature range can not be properly made on account of the lack of uniformity in the temperature ranges of the building tests. An inspection of the temperature diagrams given at the tops of Figs. 16 to 23 inclusive will illustrate this matter. For the same temperature range, the building tests show higher strength values at the same periods of time than do the

laboratory tests. This may be accounted for by the relatively high temperatures at the beginning of all the tests of the 1910–11 Series. These high temperatures were the result of the heat furnished by the salamanders, which

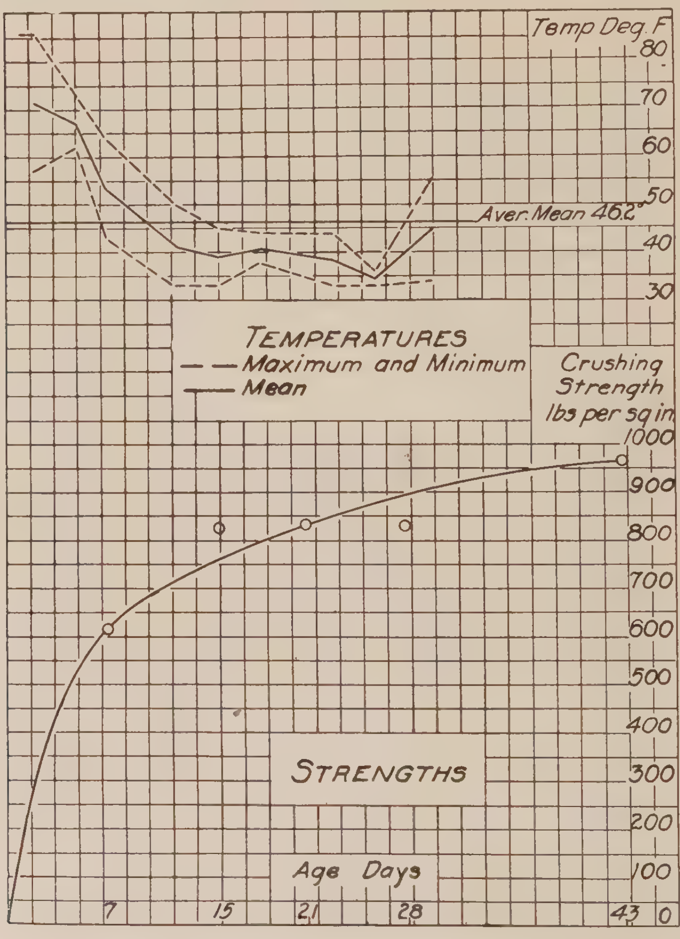

FIG. 19.—SET V, GROUP II, 1910–11 SERIES.

were kept burning during the pouring of the concrete and for several days thereafter. In the laboratory tests, the specimens of Group I of the 1913 Series were made under the storage temperature and the specimens of Group II of the 1913 Series and of Group III of the 1914 Series were

made at a normal temperature and as soon as practicable moved to their respective storage temperatures. So it is evident that all of the specimens of the laboratory series attained their respective storage temperatures within a comparatively short time after the initial set of the concrete occurred. An inspection of Figs. 19 and 2 will clearly show the high rate of increase of

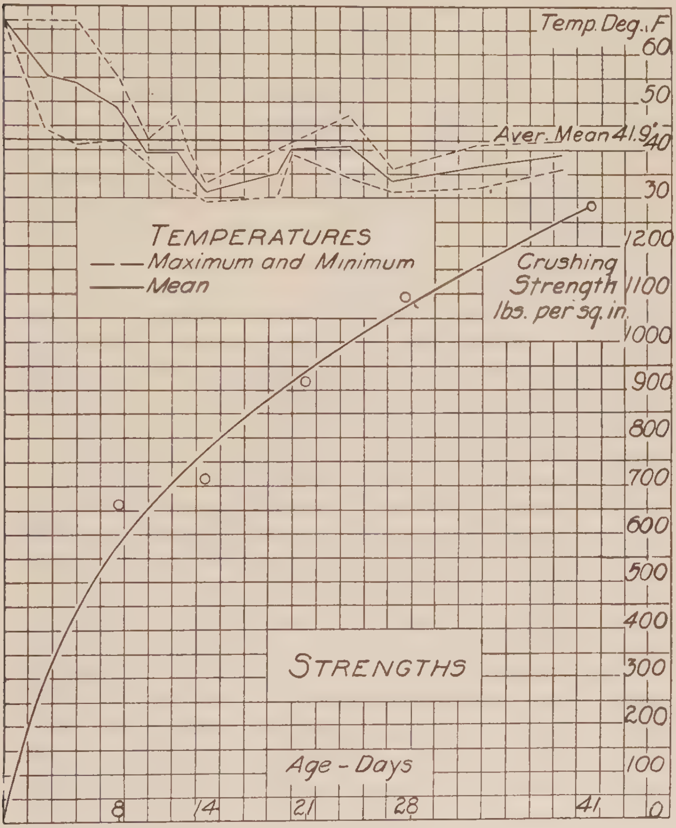

FIG. 20.—SET W, GROUP III, 1910–11 SERIES.

strength gained at high temperatures during the first few days of storage as compared with the lower rate of increase of strength under a temperature slightly higher than the average storage temperature during the same period of time. This comparison clearly emphasizes the necessity for the maintenance of the temperature of the concrete for the first few days at a point well above freezing.

Relation of Strength and Temperature.—The ranges of temperature for the 1910–11 Series are from 41.9° to 55° F. The range of temperature for each set of this Series was determined from observations of the maximum and minimum thermometer at intervals of a few days. Hence an accurate

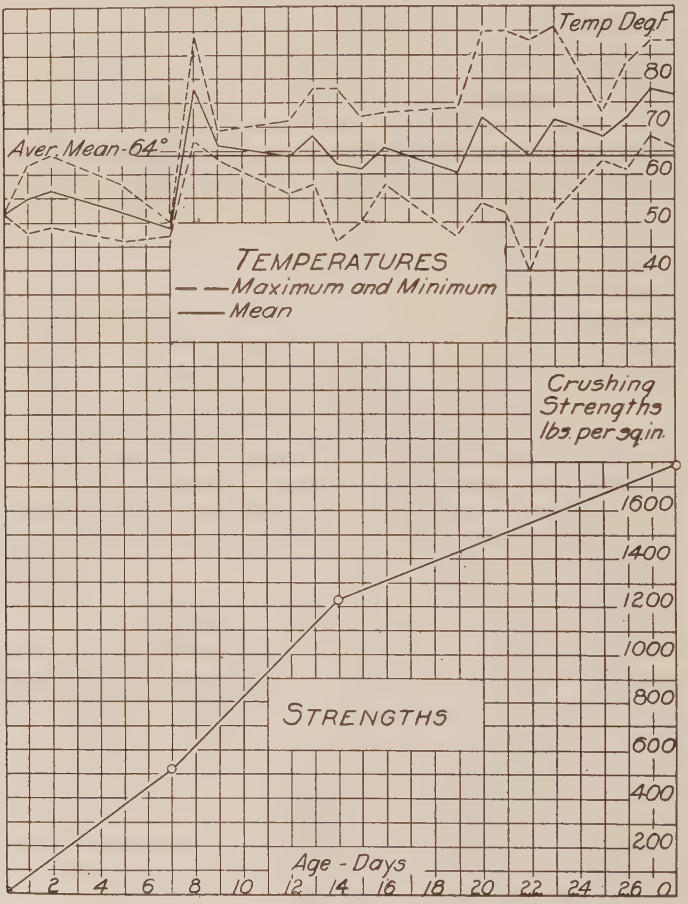

FIG. 21.—SET X, GROUP IV, 1911–12 SERIES.

determination of the temperature conditions is impossible, but the temperatures were assumed to vary uniformly between observed values to determine the values on days when no readings were taken. The details of the temperatures for the 1910–11 Series are given in Tables XXIV, XXV, XXX, XXXI and XXXII and Figs. 16 to 20 inclusive.

An inspection of Tables XXIV, XXV, XXX, XXXI and XXXII shows the variation of strength with temperature for the 1910–11 Series of tests. The last three columns of each table gives the values of strength for different ages based on the following conditions:

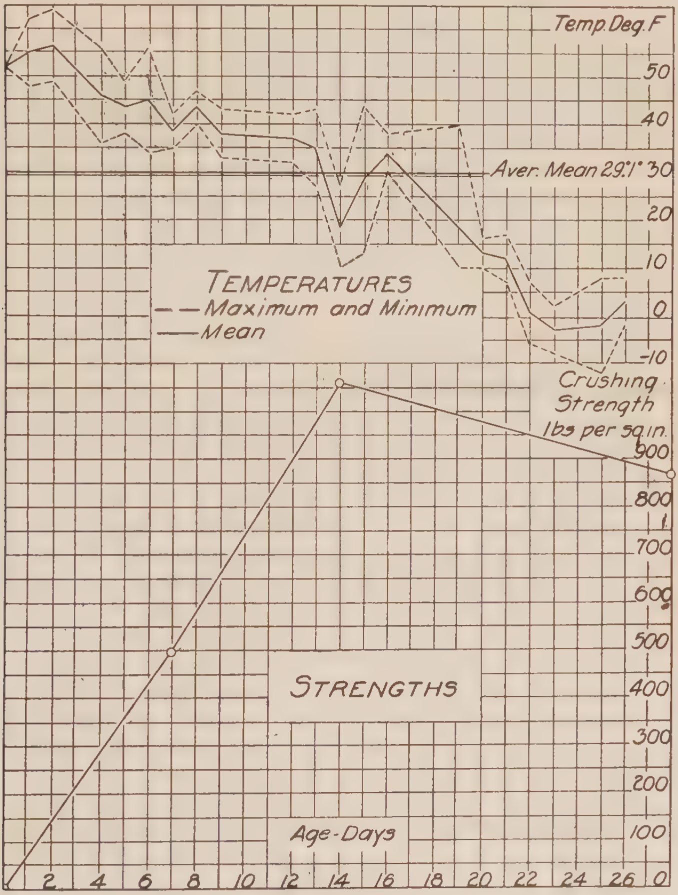

FIG. 22.—SET Y, GROUP IV, 1911–12 SERIES.

(1) The column headed "Average Strength from Test" gives the actual test values.

(2) The column headed "Average Strength from Curve" gives values taken from the curve which was drawn through the average of all the observed values.

(3) The figures in the column headed "Average Strength from Fig. 14" are values taken from Fig. 14 and correspond to the temperature values given in the column headed "Average Mean Temperature to Date."

For example, the average strength values for Set W of Group III of the

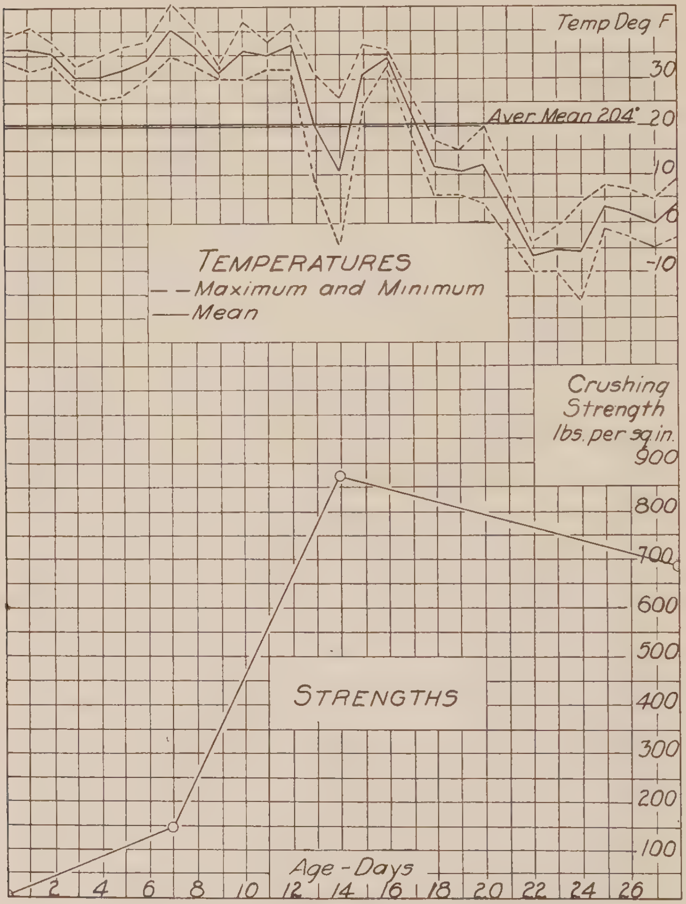

FIG. 23.—SET Z, GROUP IV, 1911–12 SERIES.

1910–11 Series of building tests are given in Table XXXIII as 659, 716, 917 and 1092 lb. per sq. in. for ages of 8, 14, 21 and 28 days, respectively. These values are stated in the column headed "Average Strength from Test" in Table XXXII. These values were plotted in Fig. 20 and the curve shown was drawn through the points so as to represent graphically the relation of

TABLE XXIV.—TEMPERATURE AND STRENGTH.

Set R, Group I, 1910–11 Series.

Age, days.	Maximum Temperature.	Mean Temperature.	Minimum Temperature.	Average Mean Temperature to Date.	Average Strength from Test, lb. per sq. in.	Average Strength from Curve, lb. per sq. in.	Average Strength from Fig. 14, lb. per sq. in.
0	65	65	65	65	....	....	....
1	..	67	..	....	....	....	....
2	..	69	..	....	....	....	....
3	77	71	65	68	....	....	....
4	73	69	64	68	....	....	....
5	..	63	..	....	....	....	....
6	64	57.5	51	66	....	....	....
7	.	54	.	....	....	....	....
8	..	51	..	63	957	900	700
9	..	48	..	....	....	....	....
10	..	45	..	....	....	....	....
11	51	42.5	34	61	....	....	....
12	..	44	..	....	....	....	....
13	55	47	39	59	....	....	....
14	63	58	53	59	1139	1100	950
15	..	61	..	....	....	....	....
16	..	64	..	....	....	....	....
17	72	66.5	61	59.6	....	....	....
18	..	64	..	....	....	....	....
19	72	61	50	59.7	....	....	....
20	53	49.5	46	58.7	....	....	....
21	..	47	..	56.0	1224	1285	1080
22	48	45.5	43	57.5	....	....	....
23	..	46	..	....	....	....	....
24	..	47	..	....	....	....	....
25	..	48	..	....	....	....	....
26	51	49.5	48	57.1	....	....	....
27	52	50.5	49	56.9	....	....	....
28	..	51	..	....	....	....	....
29	..	52	..	55.1	1380	1480	1210
30	..	53	..	....	....	....	....
31	58	54.5	51	56.7	....	....	....
32	..	....	..	....	....	....	....
33	..	....	..	....	....	....	....
34	52	48.5	45	56.1	....	....	....
35	..	....	..	....	....	....	....
36	..	....	..	....	....	....	....
37	..	....	..	....	....	....	....
38	..	....	..	....	....	....	....
39	..	....	.	....	....	....	....
40	51	45.5	40	55.4	1823	1750	....

Average Mean 55°.0 F.

Table XXV.—Temperature and Strength.

Set S, Group I, 1910-11 Series.

Age, days.	Maximum Temperature.	Mean Temperature.	Minimum Temperature.	Average Mean Temperature to Date.	Average Strength from Test, lb. per sq. in.	Average Strength from Curve, lb. per sq. in.	Average Strength from Fig. 14, lb. per sq. in.
1	63	58	53	58	....	....	....
2	..	61	..	....	....	....	....
3	..	64	..	....	....	....	....
4	72	66.5	61	62.3	....	....	....
5	..	64	..	....	....	....	....
6	72	61	50	61.8	....	....	....
7	53	49.5	46	58.7	....	....	....
8	..	47	..	52.2	780	760	600
9	48	45.5	43	56.1	....	....	....
10	..	46	..	....	....	....	....
11	..	47	..	....	....	....	....
12	..	48	..	....	....	....	....
13	51	49.5	48	55.0	....	....	....
14	52	50.5	49	54.3	1068	1060	910
15	..	51	..	....	....	....	....
16	..	52	..	....	....	....	....
17	..	53	..	....	....	....	....
18	58	54.5	51	54.4	....	....	....
19	..	52	..	....	....	....	....
20	..	50	..	....	....	....	....
21	52	48.5	45	53.7	....	....	....
22	..	48	..	....	....	....	....
23	..	47	..	....	....	....	....
24	..	46	..	....	....	....	....
25	..	46	..	....	....	....	....
26	..	45	..	....	....	....	....
27	51	45.5	40	52.9	....	1450	1180
28	..	....	..	....	....	....	....
29	..	....	..	....	....	....	....
30	..	....	..	....	....	....	....
31	..	....	..	....	....	....	....
32	..	....	..	....	....	....	....
33	..	....	..	....	....	....	....
34	46	44	42	....	....	....	....
35	..	....	..	....	....	....	....
36	..	....	..	....	1499	1630	....
37	..	....	..	....	....	....	....
38	..	....	..	....	....	....	....
39	49	47	45	....	....	....	....
40	..	....	..	....	....	....	....
41	..	....	..	....	....	....	....
42	46	40.5	35	....	....	....	....
43	..	....	..	....	1885	1760	....

Average Mean 50°.8 F.

Table XXVI.—Compressive Strength Tests.

Set R, Group I, 1910–11 Series. 8 x 16 in. Cylinders.

Specimen Number.	Age, days.	Weight, lb.	Height, in.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.
1	..	69.5	16.0	8.0	49 350	982	....
2	8	69.0	16.0	8.0	46 300	921	957
3.	..	69.0	16.0	8.0	48 600	967	...
4	..	68.5	16.0	8.0	58 250	1159	....
5	14	68.0	16.0	7.875	47 750	980	1139
6	..	69.0	16.0	7.875	62 300	1279	....
7	..	68.75	16.0	8.0	56 950	1133	....
8	21	69.0	16.0	8.0	58 450	1163	1224
9	..	69.0	16.0	8.0	69 100	1375	....
10	..	....	16.0	7.875	62 300	1280	....
11	29	69.25	16.125	8.0	70 100	1395	1380
12	..	68.25	16.125	7.875	71 420	1466	....
13	..	68.0	16.0	7.938	72 420	1464	....
14	42	69.5	16.0	8.0	99 600	1981	1823
15	..	68.0	15.875	8.0	101 680	2023	....

Table XXVII.—Compressive Strength Tests.

Set S, Group I, 1910–11 Series. 8 x 16 in. Cylinders.

Specimen Number.	Age, days.	Weight, lb.	Height, in.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.
1	..	69.5	16.0	8.0	42 700	849	....
2	8	69.0	16.0	8.0	28 000	557	780
3	..	67.5	16.0	8.0	46 900	933	....
4	..	68.0	16.0	8.0	32 780	653	...
5	14	68.0	16.0	8.0	59 520	1184	1068
6	..	67.0	16.0	8.0	68 800	1368	.
7	..	68.0	15.875	8.0	67 130	1335	....
8	36	67.0	16.0	8.0	93 000	1850	1499
9	..	68.0	16.0	8.0	66 000	1313	....
10	..	67.0	15.875	7.938	101 450	2050	....
11	43	68.5	15.813	8.0	85 430	1699	1885
12	..	66.25	15.688	8.0	95 820	1906	....

Table XXVIII.—Compressive Strength Tests.

Set T, Group II, 1910–11 Series. 8 x 16 in. Cylinders.

Specimen Number.	Age, days.	Weight, lb.	Height, in.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.
1	..	70.00	16.0	7.875	12 700	261	....
2	10	69.00	15.875	8.0	28 000	557	424
3	..	70.25	16.0	7.875	22 100	454	....
4	..	69.5	16.0	8.0	23 450	467	....
5	14	70.0	16.0	8.0	16 000	318	367
6	..	70.0	16.0	8.0	15 850	315	....
7	..	70.0	16.0	8.0	48 580	966	....
8	22	68.5	16.0	8.0	50 820	1011	1004
9	..	69.0	16.0	8.0	52 000	1034	....
10	..	70.25	16.125	7.938	33 000	667	....
11	30	70.5	16.0	8.0	27 450	546	783
12	..	70.0	16.0	8.0	57 150	1137	....
13	..	69.75	16.0	8.0	52 000	1034	....
14	43	69.75	16.0	8.0	42 300	842	912
15	..	68.5	16.0	8.0	43 180	859	....

Table XXIX.—Compressive Strength Tests.

Set V, Group II, 1910–11 Series. 8 x 16 in. Cylinders.

Specimen Number.	Age, days.	Weight, lb.	Height, in.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.
1	..	67.5	15.875	7.875	29 950	607	....
2	7	69.0	16.0	7.875	29 050	596	615
3	..	67.75	15.75	8.0	32 250	642	....
4	..	68.5	15.75	8.0	38 600	768	....
5	15	68.0	16.0	7.938	39 150	791	824
6	..	69.0	16.0	7.875	44 450	913	....
7	..	67.0	15.813	7.938	41 270	834	....
8	21	68.25	15.813	7.938	43 130	872	831
9	..	67.5	15.875	7.938	39 000	788	....
10	..	68.75	16.0	8.0	38 400	764	....
11	28	69.0	16.0	7.875	40 750	837	829
12	..	70.5	16.0	8.0	44 490	885	....
13	..	68.5	15.875	7.938	39 000	788	....
14	43	69.0	16.0	7.938	51 050	1032	964
15	..	68.0	15.75	8.0	53 860	1071	....

Table XXX.—Temperature and Strength.

Set T, Group II, 1910–11 Series.

Age, days.	Maximum Temperature.	Mean Temperature.	Minimum Temperature.	Average Mean Temperature to Date.	Average Strength from Test, lb. per sq. in.	Average Strength from Curve, lb. per sq. in.	Average Strength from Fig. 14, lb. per sq. in.
0	..	....	..	....	....	....	....
1	..	52	..	....	....	....	....
2	58	52	46	52	....	....	....
3	..	48	..	....	....	....	....
4	..	44	..	....	....	....	....
5	50	40.5	31	46.3	....	....	....
6	50	40.5	31	44.3	....	....	....
7	..	40	..	....	....	....	....
8	..	40	..	....	....	....	....
9	..	39	..	....	....	....	....
10	..	39	..	43.5	424	390	610
11	..	38	..	....	....	....	....
12	42	38	34	42.7	....	....	....
13	..	45	..	....	....	....	....
14	72	53	34	44.8	367	520	780
15	50	43	36	44.5	....	....	....
16	..	45	..	....	....	....	....
17	..	47	..	....	....	....	....
18	..	49	..	....	....	....	....
19	..	51	..	....	....	....	....
20	59	54.5	50	45.9	....	....	....
21	..	52	..	....	....	....	....
22	57	50	43	46.4	1004	745	940
23	..	48	..	....	....	....	....
24	..	46	..	....	....	....	....
25	..	44	..	....	....	....	....
26	..	42	..	....	....	....	....
27	50	41.5	33	45.8	....	....	....
28	..	41	..	....	....	....	....
29	..	40	..	....	....	....	....
30	45	39	33	45.2	783	860	1050
31	..	....	..	....	....	....	....
32	..	....	..	....	....	....	....
33	44	41	38	....	....	....	....
34	..	....	..	....	....	....	....
35	..	....	..	....	....	....	....
36	..	....	..	....	....	....	....
37	..	....	..	....	....	....	....
38	44	38.5	33	....	....	....	....
39	..	....	..	....	....	....	....
40	..	....	..	....	....	....	....
41	36	34.5	33	....	....	....	....
42	..	....	..	....	....	....	....
43	..	....	..	....	912	945	....

Average Mean 43°.0 F.

Table XXXI.—Temperature and Strength.

Set V, Group II, 1910–11 Series.

Age, days.	Maximum Temperature.	Mean Temperature.	Minimum Temperature.	Average Mean Temperature to Date.	Average Strength from Test, lb. per sq. in.	Average Strength from Curve, lb. per sq. in.	Average Strength from Fig. 14, lb. per sq. in.
0	..	....	..	....	....	....	....
1	40	30.5	21	30.5	....	....	....
2	86	71.1	57	50.8	....	....	....
3	..	70	..	....	....	....	....
4	..	69	..	....	....	....	....
5	73	67.5	62	56.4	....	....	....
6	..	60	..	....	....	....	....
7	64	53.5	43	55.7	615	615	560
8	..	51	..	....	....	....	....
9	..	48	..	....	....	....	....
10	..	45	..	....	....	....	....
11	..	43	..	....	....	....	....
12	50	41.5	33	52.8	....	....	....
13	..	41	..	....	....	....	....
14	..	40	..	....	....	....	....
15	45	39	33	50.5	824	760	860
16	..	40	..	....	....	....	....
17	..	41	..	....	....	....	....
18	44	41	38	49.1	....	....	....
19	..	41	..	....	....	....	....
20	..	40	..	....	....	....	....
21	..	40	..	48.4	831	830	980
22	..	39	..	....	....	....	....
23	44	38.5	33	47.8	....	....	....
24	..	37	..	....	....	....	....
25	..	36	..	....	....	....	....
26	36	34.5	33	46.3	....	....	....
27	..	37	..	....	....	....	....
28	..	40	..	45.5	829	895	1050
29	..	42	..	....	....	....	....
30	56	45	34	46.2	....	....	....
43	..	....	..	....	964	965	....

Average Mean 46°.2 F.

TABLE XXXII.—TEMPERATURE AND STRENGTH.

Set W, Group III, 1910–11 Series. 8 x 16 in. Cylinders.

Age, days.	Maximum Temperature.	Mean Temperature.	Minimum Temperature.	Average Mean Temperature to Date.	Average Strength from Test, lb. per sq. in.	Average Strength from Curve, lb. per sq. in.	Average Strength from Fig. 14, lb. per sq. in.
0	..	60	..	....	....	....	....
1	..	58	.	....	....	....	....
2	..	57	..	....	....	....	....
3	67	56	44	56	....	....	....
4	..	55	..	...	....	....	....
5	67	54	41	55	....	....	....
6	..	52	..	....	....	....	....
7	..	50	..	....	....	....	....
8	55	48	42	52.7	659	590	600
9	..	44	..	....	....	....	....
10	42	39.5	37	49.4	....	....	....
11	..	39.5	.	....	....	....	....
12	47	39.5	32	47.4	....	....	....
13	39	35	31	45.3	....	....	....
14	33	31	29	43.3	716	770	780
15	..	31	..	....	....	....	....
16	..	32	..	...	....	....	....
17	..	33	..	..	....	....	....
18	..	34	..	....	....	....	....
19	40	35	30	42.2	....	....	....
20	41	40	39	42	....	....	....
21	..	40	..	43.8	917	940	900
22	..	40	..	....	....	....	....
23	..	40	..	...	....	....	....
24	47	40.5	34	41.8	....	....	....
25	..	38	..	....	....	....	....
26	..	36	..	....	....	....	....
27	36	33.5	31	41.0	....	....	....
28	..	34	..	43.6	1092	1075	1040
33	41	36.5	32	....	....	....	....
39	42	39	36	....	....	....	....
41	..	....	..	....	1286	1285	....

Average Mean 41°.9 F.

Table XXXIII.—Compressive Strength Tests.

Set W, Group III, 1910-11 Series. 8 x 16 in. Cylinders.

Specimen Number.	Age, days.	Weight, lb.	Height, in.	Average Diameter, in.	Crushing Strength, lb.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.
1	..	70.0	16.0	8.0	30 800	613	....
2	8	69.25	16.0	7.938	33 420	675	659
3	..	69.0	16.125	8.0	34 700	690	....
4	..	70.0	16.0	8.0	41 100	818	....
5	14	69.5	15.875	8.0	31 400	625	716
6	..	70.0	16.0	7.938	34 900	705	....
7	..	70.25	16.0	8.0	44 550	886	....
8	21	68.0	16.0	7.875	50 470	1036	917
9	..	69.5	16.0	8.0	41 610	828	....
10	..	69.0	16.0	7.938	49 900	1008	....
11	28	70.5	16.125	7.938	55 450	1121	1092
12	..	70.0	16.0	8.0	57 700	1148	....
13	..	69.5	16.0	8.0	63 870	1271	....
14	41	70.5	16.25	7.875	70 560	1449	1286
15	..	69.0	16.0	8.0	57 260	1139	....

Table XXXIV.—Temperature and Strength.

Set X, Group IV, 1911-12 Series. 8 x 16 in. Cylinders.

Age, days.	Maximum Temperature.	Mean Temperature.	Minimum Temperature.	Average Mean Temperature to Date.	Average Strength from Test, lb. per sq. in.	Average Strength from Curve, lb. per sq. in.	Average Strength from Fig. 14, lb. per sq. in.
1	62	55	48	55	....	....	....
2	64	56.5	49	55.7	....	....	....
3	..	54	..	....	....	....	....
4	..	53	..	....	....	....	....
5	58	52	46	54.5	....	....	....
6	..	50	..	....	....	....	....
7	50	48.5	47	53	518	....	550
8	89	78	67	58	....	....	....
9	69	66	63	59.3	....	....	....
10	..	65	..	....	....	....	....
11	..	64	..	....	....	....	....
12	71	63.5	56	59.9	....	....	....
13	78	68	58	59.9	....	....	....
14	78	62	46	62.2	1222	....	970
15	72	61	50	62.0	....	....	....
16	73	65.5	58	62.5	....	....	....
17	..	64	..	....	....	....	....
18	..	62	..	....	....	....	....
19	74	60.5	47	62.2	....	....	....
20	90	72	54	62.9	....	....	....
21	90	71	52	63.5	....	....	....
22	88	64	40	63.5	....	....	....
23	91	71.5	52	64.1	....	....	....
24	..	70	..	....	....	....	....
25	73	68	63	64.3	....	....	....
26	83	72	61	64.7	....	....	....
27	88	78	68	65.4	....	....	....
28	88	77	66	64.0	1792	....	1310

Average Mean 64°.0 F.

Table XXXV.—Temperature and Strength.

Set Y, Group IV, 1911-12 Series. 8 x 16 in. Cylinders.

Age, days.	Maximum Temperature.	Mean Temperature.	Minimum Temperature.	Average Mean Temperature, to Date.	Average Strength from Test, lb. per sq. in.	Average Strength from Curve, lb. per sq. in.	Average Strength from Fig. 14, lb. per sq. in.
1	62	55	48	55	....	....	....
2	64	56.5	49	55.7	....	....	....
3	..	51	..	....	....	....	....
4	56	46	36	52.5	....	....	....
5	49	43.5	38	50.3	....	....	....
6	56	45	34	49.2	....	....	....
7	42	38.5	35	47.4	497	....	480
8	47	43.5	40	46.8	....	....	....
9	43	38	33	45.7	....	....	....
10	..	38	..	....	....	....	....
11	..	37	..	....	....	....	....
12	42	37	32	44.7	....	....	....
13	43	35	27	43.8	....	....	....
14	27	18.5	10	41.5	1059	....	730
15	44	28.5	13	40.3	....	....	....
16	38	34	30	39.9	....	....	....
17	..	33	..	....	..	....	....
18	..	29	..	....	....	....	....
19	40	25	10	38.8	....	....	....
20	16	18	10	....	....	....	....
21	17	12	7	37.0	..	....	....
22	7	0.5	−6	34.7	....	....	....
23	2	−3	−8	32.5	....	....	....
24	..	−2	..	....	....	....	....
25	8	−2	−12	30.5	....	....	....
26	8	3	−2	29.1	....	....	....
28	..	....	..	....	868	....	790

Average Mean 29°.1 F.

Table XXXVI.—Temperature and Strength.

Set Z, Group IV, 1911–12 Series. 8 x 16 in. Cylinders.

Age, days.	Maximum Temperature.	Mean Temperature.	Minimum Temperature.	Average Mean Temperature to Date.	Average Strength from Test, lb. per sq. in.
1	39	36.5	34	36.5	...
2	41	36.5	32	36.5	...
3	38	35.5	33	36.0	...
4	33	30.5	28	33.3	...
5	35	30.5	26	31.9	...
6	37	32.0	27	31.9	...
7	38	34.0	30	32.9	145
8	46	40.5	35	36.7	...
9	41	37.0	33	36.8	...
10	33	31.5	30	34.1	...
11	42	36.0	30	35.0	...
12	38	35.0	32	35.0	...
13	42	37.0	32	36.0	...
14	31	20.0	9	28.0	873
15	26	15.5	5	21.7	...
16	37	30.5	24	26.1	...
17	36	34.5	33	30.3	...
18	17	11.5	6	20.9	...
19	15	10.5	6	15.6	...
20	20	12.0	4	13.8	...
21	8	2.0	−4	7.9	...
22	−4	−7.0	−10	0.4	...
23	−1	−5.5	−10	−2.5	...
24	4	−6.0	−16	−4.2	...
25	8	3.5	−1	0.4	...
26	7	2.0	−3	1.2	...
27	9	3.0	−3	2.1	...
28	12	4.5	−3	3.3	690

Table XXXVII.—Compressive Strength Tests.

Group IV, 1911–12 Series. 8 x 16 in. Cylinders.

Age 7 Days.			Age 14 Days.			Age 28 Days.		
Set.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Set.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.	Set.	Strength, lb. per sq. in.	Average Strength, lb. per sq. in.
X	412 593 549	518	X	1114 1375 1178	1222	X	1815 1760 1802	1792
Y	529 439 523	497	Y	1072 1231 873	1059	Y	886 917 801	868
Z	161 148 127	145	Z	798 998 822	873	Z	690	690

strength to age. The values of 590, 770, 940 and 1075 lb. per sq. in. given in the column headed "Average Strength from Curve" were taken directly from an inspection of the curve. The figures in the column headed "Average Strength from Fig. 14" give values of 600, 780, 900 and 1040 lb. per sq. in. for the average unit strength for an age of 8 days under an average mean temperature of 52.7° F., for an age of 14 days under an average mean temperature of 43.3° F., for an age of 21 days under an average mean temperature of 43.8° F., and for an age of 28 days under an average mean temperature of 43.6° F., respectively. These values are all taken from an inspection of Fig. 14.

Effect upon Strength of Variations in Temperature.—Set F, Group II of the 1913 Series of the laboratory tests and Sets Y and Z, Group IV of the 1911–12 Series of the building tests show very clearly the effect of variations of temperature upon the strength of concrete. The variations include a range of temperatures below and above the freezing point of 32° F.

A study of the results of Sets F, Y, and Z shows that concrete stored under certain limited freezing temperatures increases gradually in strength at a rate depending upon the initial temperature of the materials and the temperature during the early period of the setting of the concrete. The higher these latter temperatures the greater the rate of increase in strength during the early part of the subsequent storage period. Successive temperatures above and below freezing cause alternate freezing and thawing and if continued freezing temperature follows, the concrete will show a gradual loss of strength due to the freezing of a thickness of the surface and thus weakening the specimen to the extent of the area of this frozen cylinder. However, if continued temperatures above freezing follow these fluctuations, the concrete will set again and show a gradual increase in strength.

IV. Summary.

General Conclusions.—The tests cover a wide range of temperature conditions, the average temperature varying from 20.4° to 90.6° F. The results give significant information concerning the effect of age and temperature upon the strength attained by concrete. It is believed the following general conclusions are justifiable:

(1) Under uniform temperature conditions, there was an increase of strength with age within the time limits of the tests. For any temperature the rate of increase of strength decreases with the age of the specimen; and this rate of increase is less correspondingly at the lower temperature conditions. For the specimens tested, under normal hardening temperature conditions of from 60° to 70° F., the compressive strength of the concrete subjected to a uniform temperature at the age of 7, 14 and 21 days may be taken as approximately 50, 75 and 90 per cent of the strength at 28 days respectively. For lower temperatures the percentage values are less; and for higher temperatures the percentage values are greater. The relation between the percentage values at the ages of 7, 14, 21 and 28 days is nearly the same for temperature conditions from 30° to 70° F. The values for the lower temperatures should be used with caution.

(2) It is evident that if the concrete is to acquire a reasonable self-sustaining or a load-bearing strength in a short time (conditions which ordinarily obtain on building work), it is necessary to place the concrete under the most favorable conditions and maintain these conditions during the first few days. Concrete which is protected and maintained at a temperature of from 60° to 70° F. will at the age of one week have practically double the strength of the same material which is kept unprotected at a low temperature of from 32° to 40° F. Under freezing temperature conditions the materials should be heated so that the concrete will have an average temperature of from 60° to 70° F., and the concrete in place kept under an air temperature of not less than 45° F. by artificial heat during the first week. This provision for favorable temperature conditions avoids the well-known injurious effect of the freezing of the water in the concrete, and also the deteriorating effect of the alternate freezing and thawing of the concrete.

(3) Figs. 14 and 15 may be used to determine the representative strength of concrete similar to that used in these tests, for various temperature conditions and for ages up to 28 days. These diagrams may be used with a fair degree of approximation to ascertain the relative strengths which concrete of ordinary practice may be expected to attain at the different temperatures. It should be noted that generally in this investigation the specimens were stored under temperatures which were nearly uniform during the whole storage period. In Set F the variations in temperature include a number of alternations above and below the freezing point and the specimens were seriously injured. The results accord with the well-known effect of freezing and thawing upon green concrete.

A FURTHER DISCUSSION OF THE STEEL STRESSES IN FLAT-SLAB FLOORS.*

By Henry T. Eddy.†

1. It appears to the writer that the relations which exist between the stresses in reinforcing rods as found by tests of flat slabs and the stresses as calculated by beam theory from the applied bending moments have not in general been sufficiently well known and understood by designers of flat slabs, and that those who have known what the facts in the case are have usually been somewhat at a loss as to their correct explanation.

It is with the hope of showing more clearly than heretofore has been done precisely what these relations are, and of explaining them, if possible, that the present paper is prepared in the firm conviction that there is no such inexplicable difference between correct theoretical conclusions derived from beam theory and the results of tests, as has been too often assumed.

2. The first question that arises in applying beam theory to slabs is to discover what limitations, if any, must be observed in making such application.

The beams here considered are strips of greater or less width in the slab, and the ordinary theory of beams evidently cannot be correctly applied to any strip which has vertical shearing stresses acting on its sides, unless such stresses be considered as part of the loading of the strip, since they have the effect of changing the amount of loading applied to the strip, for beam theory proper has to do with no loading except that resting immediately upon the beam. In order to treat any strip which has vertical shears acting on its sides or edges, it will be necessary to include these shears as part of the loading.

Now, in a uniformly loaded slab, the only sections across which no shears exist are the lines joining the column centers and the lines across the panels at mid-span which pass through the panel centers. Consequently the only strips in slabs not parallel to the diagonals to which it is permissible to apply a beam theory which will make the stresses in the strips depend upon their individual loads are strips bounded by the sides of the panels or by lines half way between the sides, any such strips thus having a width of some integral multiple of half a panel width. Any attempt to treat slab stresses on the basis of beam theory in which the beam strips have boundaries other than those just mentioned must necessarily involve errors of greater or less magnitude, unless corrected for vertical shears at the edges of the strips.

It is a fact, however, that beam theory is a correct method of computing the total vertical shears and the total applied bending moments in the right cross-sections of any strip of a uniformly loaded slab of indefinitely large dimensions in all directions, in case the strip is bounded by any parallel pairs

* See paper No. 1305 by the present writer entitled "Steel Stresses in Flat Slabs," *Trans. Am. Soc. C. E.*, Vol. LXXVII, pp. 1338 to 1453, inclusive (1914).

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of the above-mentioned lines, and will be very approximately correct for a slab whose outside width and length does not extend beyond a few panels, but these are the only subdivisions of a slab that can be so treated.

Any slab in which some of the panels are loaded differently from others will need to have the vertical shears at the edges of even these strips considered and allowed for in correctly applying beam theory to them.

Another reason for the limitation of strips in slabs uniformly loaded to widths that are multiples of the half span, arises from the fact that neither the vertical shears nor the applied bending moments are uniformly distributed across any right section of a strip, nor across any one section in the same manner as across any other which is differently situated in the same panel. The truth of this depends upon the fact that the slab rests upon separated supports which control the distribution of the shears. This varying distribution of shears and moments introduces certain twisting actions into these strips that prevent narrow strips from having each of them an independent beam action.

3. But the statements just made as to the correctness of beam theory for finding applied bending moments in strips does not justify the conclusion that the reinforcement in those strips affords a resistance that acts in the same manner as it does in beams, for the relations that exist between the applied bending moments and the stresses in the steel in slabs are very different from those found in beams, and are such as to render the steel in slabs much more effective than in beams, as will appear later.

Let it be assumed, according to beam theory, that the sum total of half the numerical values of the statical bending moments applied at the ends of any span, L, of a beam strip plus that applied at mid-span arising from a total uniformly distributed load W amounts to $WL/8$, whether the span is a simple one with end supports, or is continuous, or partially so, and this without regard to the loading or lack of it on other spans of the beam strip. The truth of this assumption may be established as follows:

Let M_0 designate the applied bending moment at mid-span.
M_1 and M_2 the moments at the ends.
S_1 and S_2 the numerical values of the shears at the ends.

Then $S_1+S_2=W=wL$ is the total uniformly distributed load of intensity w per unit of length of the span L. Then the applied bending moment at mid-span may be expressed in either of the forms:

$$M_0=M_1+\tfrac{1}{2}\,S_1L-WL/8.$$
$$M_0=M_2+\tfrac{1}{2}\,S_2L-WL/8.$$
$$\text{Hence } 2M_0=M_1+M_2+\tfrac{1}{2}\,(S_1+S_2)\,L-WL/4,$$

or substituting the value of S_1+S_2 just given, we have

$$M_0-\tfrac{1}{2}\,(M_1+M_2)=WL/8,$$

which is the algebraic expression of the theorem which was to be established, as is evident when it is noticed that the end moments over the supports are necessarily negative.

It should be further noticed that this theorem is valid and holds true both crosswise and lengthwise of the slab independently and in each of the two directions separately into which it may be divided by strips at right angles to each other drawn parallel to the rows of columns which extend across and along the floor.

The constant $WL/8$, which may with propriety be called the applied bending moment per span, will be used to furnish the basis for the numerical comparisons that follow in this paper.

In applying this theorem to beam strips in flat slabs which are one panel wide and uniformly loaded with a total load of W per panel, it will make no difference in the constant $WL/8$ whether the strip be taken to be so located that the two halves of the strip lie side by side in the same panel so as to have the supports at the four corners of the panel or be so located as to form adjacent halves of adjacent panels, so as to have supports along the middle line of the strip, but in calculating the moment of resistance of strips a closer estimate can frequently be made for the limited areas subjected to test loads when the strip is so taken as to have supports along its middle line.

The peculiar advantage which this theorem has over others for the discussion of test data lies in the fact that it is wholly independent of the presence and distribution of any loading other than that within the one span under consideration, except insofar as it may be necessary to make allowance for vertical shears at the edges. This is a very important advantage in the discussion of tests where the area to which loading is applied is necessarily limited to a few of the panels of the entire floor, whereas ordinary design assumes that all the panels are equally loaded. This theorem permits, also, the discussion for purposes of design of the case of part only of the panels subjected to load, a fact which opens a wide field of inquiry, while the methods that have reference merely to the case of all panels equally loaded leave much to be desired.

4. It is usually necessary in floor construction to furnish the columns with enlarged capitals in order to provide suitable resistance to vertical shearing stresses at the supports. These caps have a diameter of not less than $0.2\ L$, a fact which may reduce the effective span from L to $0.8\ L$, at least for those areas of the slab which lie between the supports, but without making any change in W that need be considered. The constant value of the effective applied bending moment each way in each panel may then be reduced from $WL/8$ to $0.8WL/8 = WL/10$, in case the resistance to bending is concentrated in belts of steel which all pass over the stiff heads of the columns and none of it crosses the side of the panel between these heads. But in ordinary cases it will be better to assume a somewhat smaller reduction than this. Except in cases where the loading W is increased by shears at the edges of the strip, we shall consequently assume the effective value of the constant applied moment per span to be reduced from $WL/8$ to $WL/9$ by the effect of the cap.

In case of perfect continuity, as, for example, in a continuous plate of metal, where the distribution of the applied bending moment along the cross-section between supports is more nearly uniform than in a slab with belts of

steel crossing over the columns, the effective span even with enlarged caps is not reduced by so large a percentage as in the slab.

In the case of perfect continuity of a uniform metal plate without enlarged supports the constant applied moment per span, $WL/8$, is subdivided between mid-span and ends in such a way that

$$M_0 = -\tfrac{1}{4}(M_1+M_2) \text{ or } M_0 = WL/24$$

is the applied moment across the center line of a panel, and if

$$M_1 = M_2, \text{ then } -M_1 = WL/12,$$

which is the total applied moment across a section between supports. That is to say, with all panels uniformly loaded $-M_1 = 2M_0$, and the applied moment at mid-span is half that at either end, so that their numerical ratio is 1 to 2. Any reduction of load on adjacent spans will increase M_0 and decrease the numerical values of M_1 and M_2.

In case, however, where the effective span is reduced to $0.8L$ by the cap, and the magnitude of the total constant applied moment per span is thereby reduced from $WL/8$ to $WL/10$, but the numerical value of the moment at each end is still twice that at mid-span by reason of continuity due to the large amount of resisting steel that crosses over the caps, it is evident that the applied moments at the ends and mid-span will be reduced in approximately the same ratio, so that the applied moment at mid-span would be only $WL/30$ instead of $WL/24$, and that at the edge of the cap only $WL/15$ instead of $WL/12$.

The distribution of these applied moments along the sections at mid-span and between column centers respectively, will not in general be uniform, but will be controlled by the relative rigidities of the resisting material across the different portions of these cross-sections. As an illustration of this, take the case of the flat steel plate in the crown sheet of a boiler with stay-bolts as supports. The holes through which these bolts pass weaken the plate somewhat, so that an applied bending moment of somewhat less than $WL/12$ takes effect between the bolts and somewhat more than $WL/24$ on each mid-section. The moment first mentioned is all distributed along the section between the bolt holes, because none of it whatever can be resisted by that part of the section which is cut away by the bolt hole itself, a fact which illustrates how the rigidity, or the lack of it, at any part of the cross-section controls the distribution of the moments along it. It also shows how the distribution of the moment along the end section is independent of that at mid-span, although the relative total magnitude of the moments at mid-span and ends depends upon the relative magnitudes of the total rigidities at these sections.

A distribution of applied moments very different from that just considered takes place in a two- or four-way slab where most of the tensile resistance at the ends is found in the belts of rods that cross over the column heads, in a width not greater than $L/2$. The bending moment of $WL/15$, say, applied at the cross-section between column centers will be almost entirely applied in a width of $L/2$ across the column heads and a negligible amount for a width of $L/2$ midway between.

5. It is common among designers and in engineering literature to assume in the case of the flat slab that an amount of reinforcement is required in all the belts that cross in 180 deg. around the column center, sufficient to resist safely a total applied bending moment of $WL/15$ or $WL/12$ when calculated according to beam formulas. It will be noticed that this, which is without justification in beam theory, provides steel to resist only one-half of the bending moments actually applied in 180 deg. around the column center, as has already been shown, and yet steel to resist stresses of this reduced amount is not only amply justified by practice, but tests show that the actual stresses are far less even than those so computed.

It is not known that there has been any attempted proof of the correctness of this assumption which has crept into use in the engineering profession other than possibly the vague assertion that one-half of the load may be taken to be carried to the supports by one set of bending moments and the other half by another set at right angles to those first mentioned. Any such assertion is, however, clearly a fallacy, because the fundamental action by which the load is transmitted to the supports is by vertical shear in the slab along the most direct lines that the rigidity of the slab in its resistance to this shear will permit. This transmission of the load by vertical shear in the slab is to be regarded as its primary function. It is a function that is not disturbed, altered or interfered with by the subdivision of the total applied bending moments per span between the ends and mid-span. The applied bending moments are to be regarded merely as a by-product, and as filling the secondary or auxiliary function of enabling the vertical shears to perform their primary function of transferring the load on the panel and causing it to rest upon the supports. As has already been seen, the bending moment at one cross-section takes the place of that at another, so that the sum total is constant. The actual subdivision of the total applied bending moment per span, which is controlled by the relative resistances at the edge and mid-span sections of the panel, does not affect the distribution of vertical shears.

In a slab the total vertical shear around a support is distributed circumferentially according to the distribution of the rigidities of the vertical resistances around the support and is independent of the distribution of the bending moments, except insofar as it happens that the rigidity of identically the same resisting material in the structure acts to resist both shear and bending at the same time. The assumption of a total bending moment of $WL/12$ or $WL/15$ in 180 deg. around a support, instead of twice that amount in accordance with the principles of statics, cannot, therefore, be justified on any such basis as this. So far as is known, the only other suggestion that has been made as a basis for assuming stresses in the steel only one-half those due to the applied bending moments is the assumption that direct tensile stresses in the concrete act in conjunction with those in the steel, and that these both assist each other in resisting the bending moments. But such direct tensile stresses in concrete are not regarded as admissible by any one, because they are not to be relied upon permanently.

6. Let us, therefore, turn to other considerations in order to arrive at a correct determination of the relation of the steel stresses to the applied bend-

ing moments. The magnitudes of the total applied bending moments across sections at mid-span and sections between columns in a uniformly loaded slab have already been stated, and it remains to find out theoretically what stresses these applied moments should produce in the steel that crosses these sections, and then compare them with test results in the data at our disposal so as to establish the correctness of our theoretical deductions.

According to beam theory, when steel rods are used to resist tensile stresses their longitudinal resistance is alone called into play, but were it possible instead to use a sheet of metal to resist at once both the longitudinal and lateral tensile stresses which act at right angles over columns and at panel centers, such as arise from the action of two applied bending moments at right angles to each other, then this single sheet would resist both of these moments at once without more weight of metal than would be required to resist either one of the applied moments separately. Were the belts of reinforcing rods over a column each to act separately and independently and so merely longitudinally, it is evident that twice the weight of steel would be required that would be necessary in the form of a sheet which is effective both laterally and longitudinally at once.

Now the fact is that when belts of rods which lie one across the other are embedded in a bulky concrete matrix, the matrix so ties the crossed rods together as to replace the connections of a continuous sheet and causes the steel to act in conjunction with the concrete like a continuous sheet of steel of the same total weight as the crossed belts. This effect is produced by the bond-shear called into play to prevent the concrete from sliding along the surfaces of the steel rods. It is evident that so long as the steel does not slip in the concrete, as in fact it does not in slabs, there must be some possible or supposible degree of rigidity in the matrix and its connections that would afford a resistance sufficient to take the place of the connections that would transform the belts into a single sheet so far as its resistance to bending is concerned.

It is, therefore, merely a question of experiment to find out whether the combination of concrete and steel does afford such resistance, as in fact it is found to do. This fact would, therefore, justify the common assumption, previously stated, of calculating a total cross-section of reinforcement in 180 deg. about each column center no more than sufficient to resist $WL/12$ or $WL/15$ when, in fact, the applied moments together amount to twice that.

This, however, is a statement of one part only of the relations necessarily involved in the case in hand, for since the steel does not slip in the concrete such changes of stress and deformation as occur along each rod are accompanied by equal stresses and deformations in the concrete in contact with it. It appears, therefore, that the total amount of work expended by the load in deflecting a slab will be one-half of it expended in deforming the concrete and the other half in deforming the steel, of which latter amount one-half, or one-fourth of the entire amount, will be expended in deforming the steel in one direction and one-fourth on the steel at right angles thereto.

It appears, therefore, that the steel will be subjected to only one-fourth as great stresses in such a slab as would be calculated in accordance with beam theory, in which the section of the steel was computed to resist the

entire applied bending moment. According to this theory the total actual resisting moments in the steel across each mid-section of a panel of a uniformly loaded slab would be from $WL/96$ to $WL/120$, and across each section at the edge of a panel between column centers would be from $WL/48$ to $WL/60$.

But this computation has neglected to take account of the fact that longitudinal deformations are accompanied by reversed lateral deformations commonly mentioned under the head of Poisson's ratio.

If stresses in steel are to be measured by deformations, as they in fact are in all tests, then the effect of Poisson's ratio must necessarily be considered and allowed for. So far as tests go, the changes of deformation due to this cause appear to reduce the steel stresses in slabs to three-fourths of what they would otherwise be. This would make the total observed resisting moment of steel across each mid-section parallel to the edge of a panel of a uniformly loaded slab to lie between $WL/128$ and $WL/160$, and that across the section at the edge between column centers to lie between $WL/64$ and $WL/80$. It will be observed that, according to this amended theory, the applied bending moments amount at least to $4 \times \frac{4}{3} = 5\frac{1}{3}$ times those actually to be found on this theory from the elongations of the steel, and that the *ordinarily* assumed basis of design of $WL/12$ to $WL/15$ in 180 deg. around column centers would give a resistance of at least $2 \times \frac{4}{3} = 2\frac{2}{3}$ times that calculated according to this amended theory.

In case any steel stresses are omitted in calculating the resistance of a slab, as, for example, leaving out those in radial or ring rods, or in case there are direct tensile stresses in the concrete, then the figure just mentioned will be too small.

We are now prepared to consider test data in order to learn whether observations confirm the correctness of the results of the above reasoning as to the manner in which the steel resists the applied moments in slabs. It is not, however, to be expected that these results will be confirmed numerically for light loads on account of residual direct tensile stresses in the concrete, but for loads above design load, up to twice the design load, they may be expected to be in reasonable accord with observation. But whether the mechanical relation of the resisting moment to the applied moment be that which has just been outlined or not, it is evident that some definite mechanical relation exists between these two magnitudes and that they depend the one on the other, so that if it be possible so to analyze observed phenomena as to discover what their ratio is, then it may be of great practical use in the calculation of slabs, even though the explanation of that relation which has just been proposed may appear to some to be not well founded.

Well ascertained facts must be recognized, be they explicable or not. It seems at least reasonable, not to say strongly probable from the nature of the case, that the ratio between the applied moment and the moment of resistance of the steel should be practically the same in all flat slabs of similar construction, and that in fact is just what is about to be established by the results of the several tests that follow.

7. Test of the Deere & Webber Co. Building in Minneapolis,* made by A. R. Lord, November, 1910:

This slab is $9\frac{3}{8}$ in. thick and the panels are 19 ft. 1 in. by 18 ft. 8 in.; mean span, $L=226.5$ in.; design load, 225 lb. per sq. ft.; maximum test load per panel, $W=124,678$ lb., and $WL=28,239,470$ in.-lb.

In this test the uniform load extended over eight contiguous panels forming, with the exception of one corner panel, a square area of nine panels, three on a side. The measurements for deformation of steel were all in the interior and on the sides of two adjacent panels, viz., in the center panel of the nine just mentioned and the one next to it lying between two loaded corner panels, so that these two formed part of an interior tier of three loaded panels across the middle of the loading, as nearly as possible. The theorem of constant applied bending moment would apply to either of these two panels and it could be verified approximately were the observations sufficient in number and so distributed as to make it possible to do so with accuracy. Such, however, was not the case, and the best that can be done with the data given is to determine average stresses in the side and diagonal belts at mid-span and over the columns, regardless of their location, and make use of the moments so obtained. The lack of stiffness at the junction between columns and slab by reason of almost entire absence of stiffening rods, such as elbow rods, seems to have largely prevented the columns from acting integrally with the slab. We shall, therefore, take the constant applied bending moment per span in this case to be $WL/8=3,530,000$ in.-lb.

Each side belt has 12 and each diagonal belt 14 round rods $\frac{7}{16}$ in. in diameter. The measured elongations showed an average unit stress at mid-span of the side belts of $f_s=8050$ lb. The moment of resistance of a side belt across the mid-section of the panel is

$$jd_1f_sA_1=0.91\times8.5\times8050\times12\times0.15=112,080 \text{ in.-lb.}$$

The mean unit stress in the diagonal rods at the panel center was $f_s=4800$ lb.

It is assumed that belts of rods crossing each other at right angles, as do these diagonal belts, exert an equal resistance in all directions, and that the stresses at mid-span would be unchanged were the belts made parallel to the sides. The moment of resistance of the diagonal belts across the mid-section of the panel is, therefore,

$$jd_2f_sA_2=0.89\times8\times4800\times14\times0.15=71,770 \text{ in.-lb.}$$

Hence the total resisting moment of the steel across the section at mid-span $=183,850$ in.-lb.

The mean unit stress in the side belts over the column was $f_s=15,900$ lb., and in the diagonals over the cap was $f_s=18,900$ lb. Hence the sum of the resisting moments of these belts over a column $=$

$$0.83\times7.6\ (15,900\times12+18,900\times14)\ 0.15=430,900 \text{ in.-lb.,}$$

which may be taken as half the sum of the resisting moments of the steel at the two ends of the span. Hence the sum total of half the resisting moments

* See *Proc. Nat. Assoc. Cement Users*, Vol. VII, Philadelphia, 1911, and also "Concrete-Steel Construction," Eddy and Turner, p. 228.

at the ends of the span, plus that at mid-span = 615,000 in.-lb., and we find that the constant applied moment per span is 5.7 times this resisting moment, a number which somewhat exceeds the theoretical value of $5\frac{1}{3}$, as is to be expected, because several small additions to the resisting moment have been neglected, viz., those due to the resistance of the radial rods, etc., which would increase the resisting moment somewhat and make the final result somewhat less than 5.7.

A similar computation for the design load of 225 lb. per sq. ft. makes $W = 80{,}150$ lb. and $WL = 18{,}154{,}000$ in.-lb., or $WL/8 = 2{,}269{,}000$ in.-lb.

The moment of resistance of a side belt at mid-span was not more than

$$0.91 \times 8.5 \times 3000 \times 12 \times 0.15 = 41{,}770 \text{ in.-lb.}$$

and the moment of resistance of the diagonal belts at mid-span was

$$0.89 \times 8 \times 2000 \times 14 \times 0.15 = 30{,}000 \text{ in.-lb.,}$$

or a total of 71,770 in.-lb. at mid-span. Also the moment of resistance of these belts over each column was

$$0.83 \times 7.6\ (9000 \times 12 + 11{,}000 \times 14)\ 0.15 = 248{,}000 \text{ in.-lb.,}$$

giving a total resisting moment of 320,000 in.-lb., so that in this case the constant applied moment per span $WL/8$ is 7 times the resisting moment of the slab steel. The excess over the theoretical value, $5\frac{1}{3}$, is in this case necessarily greater because the loading is smaller and the direct tensile stress of the concrete, which is neglected with that of the radial rods, etc., is a much larger fraction of the total resistance.

It will be noticed that the resisting moments at mid-span and ends are not in the ratio of 1 to 2, as would be the case in a uniform continuous plate of many spans all equally loaded. But the moments at mid-span are determined from the middle panel of three loaded panels, a distribution of loading which very greatly reduces the bending moment at mid-span in the middle panel so that the end moments are much more than twice the end moments in this case. The theorem of constant moment per span, which has been here applied, is not well suited to treat the average stresses in several spans, for it is a theorem which applies to each separate span independently, as has been already stated. The agreement between it and these average results which are influenced by different panels differently is, therefore, not entirely satisfactory and will partly explain the rather excessive value of 7 last found.

8. Test of the A. J. Franks Building, Chicago, by Dr. W. K. Hatt, August, 1911.*

The thickness of this rough slab as tested was $9\frac{1}{4}$ in., which was increased to $13\frac{1}{4}$ in. over the area of a square drop head of 6 x 6 ft. at the top of each column. Each panel was 19 ft. 4 in. by 20 ft. 3 in., or 232 by 243 in.

Reinforcing rods were all of them $\frac{1}{2}$-in. rounds, 16 in each side belt or cross band, and 18 in each diagonal. All the rods of each side belt were lapped over each column for a distance of 2 ft. beyond the column center each way, and the diagonals for 6 ft. each way from column centers.

* See report of this test in *Engineering News*, April 18, 1912, and *Proc. Nat. Assoc. Cement Users*, 1912.

The design live load was 256 lb. per sq. ft., and the maximum test load on four panels at once was 312 lb. per sq. ft., while the maximum test load on two loaded panels was 624 lb. per sq. ft.

Consider first the case of the maximum test load on four panels of 312 lb. per sq. ft., and take a beam strip across the middle of the loaded area so located as to have a row of columns long its center line. The observations were such as to enable us to know the average values of the steel stresses in the direct and diagonal belts at mid-span, and also over the central column, as well as those in the direct belt across an end column; they are lacking, however, as to stresses in diagonal belts across any end column. These, however, may be assumed without much probable error, since they cannot be greater than those in the direct belt over the end column.

The computation will then be as follows for an area equal to a single panel formed of two halves of panels side by side extending half way across the middle of the four loaded panels, with a column situated at the middle of each end of the strip. For the test load of 312 lb. per sq. ft., the total panel load $W=312\times 391\frac{11}{24}=133{,}135$ lb. The mean span $L=237.5$ in., hence $WL=29{,}000{,}000$. The constant applied bending moment per span, $WL/9=3{,}223{,}000$ in.-lb. The greater stiffness of the connections joining columns and slab in this structure is regarded as reducing the effective span to $0.8L$, and this reduces the constant from $WL/8$ to $WL/9$, as previously explained. In this floor take $d_1=8.5$ in.; $d_2=8.25$ in., and $d_3=11.5$ in. Then the resisting moment of the direct belt at mid-span is

$$jd_1f_sA_1=0.91\times 8.5\times 5360\times 16\times 0.196=129{,}600 \text{ in.-lb.},$$

and that at mid-span of the diagonal belt is

$$jd_2f_sA_2=0.89\times 8.25\times 1680\times 18\times 0.196=42{,}900 \text{ in.-lb.},$$

making a total resisting moment of steel across the mid-section of the panel$=172{,}500$ in.-lb.

Again, one-half the resisting moments of the direct and diagonal belts across the center column, where both belts, owing to laps, have double the number of rods which they have at mid-span, will be found by taking the actual number of rods in these belts without laps as effective as follows:

$$0.83\times 11.5\ (3900\times 16+3400\times 18)\ 0.196=231{,}600 \text{ in.-lb.},$$

where 3900 lb. and 3400 lb. are respectively the average unit stresses in the direct and diagonal belts over the center column. Similarly, one-half the resisting moment of the steel crossing over the end column at the edge of the loaded area was

$$0.83\times 11.5\times 2500\ (16+18)\ 0.196=163{,}000 \text{ in.-lb.},$$

when it is assumed, in default of any observations, that the unit stress in the diagonal rods$=2500$ lb., since that is the average unit stress in the rods of the direct belt. Consequently half the sum of the resisting moments of the steel at the ends of the span$=394{,}600$ in.-lb., which is somewhat more than twice that at mid-span, as was to be expected.

The sum total of half the two resisting moments at the ends plus that

at mid-span = 567,100 in.-lb. The computed applied bending moment, $WL/9 = 3{,}223{,}000$, is 5.68 times this resisting moment of the steel, which, as before explained, somewhat exceeds the theoretical value of $5\frac{1}{3}$ by reason of residual tensile stresses not taken by the belts.

It is to be noted that the average unit stresses which have been employed in this test are those due to the live load only and they do not include those due to the dead load, which are included in the figures for average unit stresses as given in the original report previously referred to.

A similar calculation of test results of the Frank's Building under the design load of 256 lb. per sq. ft. is as follows:

$W = 256 \times 391\frac{1}{2}\frac{1}{4} = 100{,}000$ lb.; $WL = 23{,}800{,}000$ in.-lb.; $WL/9 = 2{,}640{,}000$ in.-lb.

Moment of resistance of steel at mid-span of direct belt was

$$0.91 \times 8.5 \times 3760 \times 16 \times 0.196 = 90{,}950 \text{ in.-lb.}$$

Moment of resistance at mid-span of diagonal belt was

$$0.89 \times 8.25 \times 830 \times 18 \times 0.196 = 21{,}170 \text{ in.-lb.}$$

Total moment at mid-span, 112,100 in.-lb. One-half the total moment of resistance over center column was

$$0.83 \times 11.5\ (3150 \times 16 + 2490 \times 18)\ 0.196 = 178{,}400 \text{ in.-lb.}$$

One-half the total moment over the end column was

$$0.83 \times 11.5 \times 2000\ (16 + 18)\ 0.196 = 127{,}450 \text{ in.-lb.}$$

Hence one-half the moment at both ends = 305,850, or somewhat more than twice that at mid-span.

The sum total of half the resisting moment at both ends plus that at mid-span = 418,000 in.-lb. The applied moment $WL/9$ is 6.3 times as much, a number not quite so much in excess of the theoretical $5\frac{1}{3}$ as that for the design load in the test of the Deere & Webber Building.

The data are insufficient to make a similar calculation for the test load of 624 lb. per sq. ft. on two panels situated diagonally to each other.

The percentage of steel in this floor is much larger than in the Deere & Webber Building, but that does not seem to affect materially the relation of the applied bending moment to the resisting moment of the steel, at least for heavy test loads.

9. Test of the Larkin Building, Chicago, by A. R. Lord.*

The slab thickness was 9 in., with an additional drop head 6.75 in. thick and 8 ft. square over each column cap integral both with column and slab, making a total thickness of about 16.75 in. at the edge of the cap. The panels are 24 ft. 2 in. by 20 ft., or $L_1 = 290$ in. and $L_2 = 240$ in.

The architect's blue print design shows the following belts of reinforcement, all of $\frac{1}{2}$-in. round rods:

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* See *Proc. Nat. Assoc. of Cement Users*, Vol. IX. Also extracts from the same in the *Cement Era*, January, 1913; *Trans. Am. Soc. C. E.*, Vol. LXXVII, p. 1368, 1914; "Concrete Steel Construction," Eddy and Turner (1914), p. 230.

For interior panels, 13 rods on each side, 26 on each long side and 19 on each diagonal.

For exterior or wall panels, 16 on each short side, 26 on each long side, and 21 on each diagonal.

The design live load was 225 to 250 lb. per sq. ft. The test load was applied in several stages. In stage 3, it amounted to 250 lb. per sq. ft., uniformly distributed over five panels, three of them exterior or wall panels and two interior panels adjacent to them. In stage 4, three panels only were loaded, two of them wall panels and one interior panel adjacent to them, with a uniform load of 415 lb. per sq. ft. Finally, in stage 5, a live load of 618 lb. per sq. ft. was placed upon two adjacent panels, one of which was a wall panel and the other an interior panel so situated that they had their short sides in common and their long sides at right angles to the wall. This last stage is the only one that lends itself to any close calculation of applied and resisting moments because of the unsymmetrical disposition of the loaded panels in the earlier stages, and the consequent uncertainty as to the magnitude of the shears and moments involved.

The stresses given in the report have reference to those estimated to be due to both the live and dead loads acting together as increased proportionally above those observed to be due to the live loads only, these observations not being reported.

The maximum test load is, therefore, taken as 738 lb. per sq. ft. to include both live and dead loads, making a total panel load of $W=738\times 20\times 24\frac{1}{6}=356{,}700$ lb.

First consider a beam strip one panel wide lying across the middle of the two loaded panels parallel to the wall, and take moments about sections perpendicular to the wall. This strip has a short side belt joining the two columns which are at the middle of each of the ends. In this case

$$L_2=240,\ WL_2=85{,}606{,}000 \text{ in.-lb., and } WL_2/9=9{,}512{,}000 \text{ in.-lb.}$$
$$\text{Take } d_1=8 \text{ in., } d_2=7.75 \text{ in., and } d_3=14 \text{ in.}$$

The resisting moment of the steel at mid-span of the side belt was

$$0.91\times 8\times 24{,}200\times 13\times 0.196=449{,}700 \text{ in.-lb.}$$

and the moment of the diagonal belt at mid-span was

$$0.89\times 7.75\times 12{,}900\times 19\times 0.196=332{,}000 \text{ in.-lb.,}$$

which gives a total resisting moment of steel at mid-span of 781,700 in.-lb.

Half the sum of the equal resisting moments at the ends of the span is the moment at one end over a column, of which that due to the direct belt of 13 rods plus 6 laps is

$$0.83\times 14\times 7300\times 19\times 0.196=316{,}700 \text{ in.-lb.,}$$

and that due to the diagonal belts of 19 rods without laps would be

$$0.83\times 14\times 10{,}800\times 19\times 0.196=468{,}200 \text{ in.-lb.}$$

But the diagonal belts have short laps, which may be estimated to increase the resisting moment of the steel in the diagonals over the columns

by 25 per cent. Hence the resisting moment of the diagonals over one column may be taken as $468{,}200 \times .25 = 582{,}250$ in.-lb., which makes the half sum of the moments at the ends $= 898{,}850$ in.-lb.

Hence the sum total of half the resisting moments at the ends plus that at mid-span $= 1{,}680{,}650$. But the numerical value of $WL_2/9$ is 5.66 times this resisting moment.

In making this calculation the vertical shears at the edges of the strip have been neglected. They might in such a case amount to some quantity not to exceed $W/8$, which would determine $WL_2/8$ as an outside limiting maximum value of the applied bending moment, instead of $WL_2/9$ which has been used. Any such increase of this constant would have the effect of making it a still larger multiple than 5.66 of the resisting moment of the steel, as might be expected from the influence of the nearby wall, which adds its resistance to that of the steel and makes the latter exert a moment which is a smaller fraction of the applied moment.

Secondly, consider a beam strip at right angles to the wall including the entire loaded area of the two panels. The applied bending moment about lines parallel to the wall are resisted by all the steel in both the side belts at the edges of the loaded area. The strip to be considered must, therefore, include both these belts. Assume the strip to have a width of $2L_2$ and to include on each side of the loaded area width of $\frac{1}{2}L_2$. It can be shown that the total vertical shear at the two edges of each panel of this strip makes an additional load of about $W/8$ per span, which, as previously stated, must be allowed for in finding the moment constant per span of this strip.

As before $W = 356{,}700$ lb., and $WL_1 = 290\ W = 103{,}443{,}000$ in.-lb. Making allowance for shear at the edges, the constant $WL_1/8 = 12{,}930{,}000$ in.-lb.

Calculate for the interior long span of this strip as follows: As shown in the report of the test, the gage lines at mid-span on one side of this area were both situated under the load, and so far as can be learned from the figures published, a mean unit shear of 8520 lb. or less should be assumed, hence the moment of resistance of the steel at mid-span of the two long side belts may be taken as

$$0.91 \times 8 \times 8520 \times 46 \times 0.196 = 560{,}200 \text{ in.-lb.,}$$

while the moment at mid-span of the diagonals was

$$0.89 \times 7.75 \times 12 \times 12{,}900 \times 19 \times 0.196 = 332{,}000 \text{ in.-lb.,}$$

making the total moment at mid-span 892,200 in.-lb.

One-half the moment of resistance of two long side belts of 23 rods and 13 laps each over the columns at the middle of the strip was

$$0.83 \times 14 \times 7000 \times 36 \times 0.196 = 575{,}000 \text{ in.-lb.}$$

One-half of the moment of two diagonal belts of 19 rods each over middle columns of strip, as previously computed, was

$$0.83 \times 14 \times 10{,}800 \times 19 \times 0.196 = 468{,}200 \text{ in.-lb.,}$$

in which no addition is required for laps because the 35-ft. long rods starting from the wall all end in a line parallel with the face of the wall and do not

perceptibly increase the resisting moment of the steel at the edge of the cap about a line parallel to the face of the wall. Hence one-half the resisting moment over the two columns at the middle of the strip was

$$575{,}000+468{,}200=1{,}043{,}200 \text{ in.-lb.}$$

One-half the resisting moment of the steel at the other end of the span may be shown by the theory of continuous beams not to exceed about one-third of that at the middle of the strip, or

$$1{,}043{,}200/3=347{,}700 \text{ in.-lb.}$$

The sum total of half the resisting moments at the ends of the span, plus that at mid-span may, therefore, be taken as 2,283,100 in.-lb. In this case the constant applied moment per span previously computed of $WL_1/8=$ 12,930,000 in.-lb. is 5.65 times this resisting moment, a number which exceeds $5\frac{1}{3}$, as before stated, because not quite all the resisting moment is furnished by belt rods alone.

10. Test of the Northwestern Glass Company Building, Minneapolis, May, 1913, by F. R. McMillan.*

This is a mushroom flat slab 8 in. thick. The interior panels are 16 x 17 ft. and the wall panels 17 x 17 ft., with 15 $\frac{3}{8}$-in. round rods in each belt without long laps over the columns. The design load was 400 lb. per sq. ft., or a total load per panel of $W=108{,}800$. Test load 7, the heaviest load upon a single interior panel, amounted to $W=183{,}000$ lb. Assume $L=200$ in., then $WL=36{,}600{,}000$ in.-lb., and $WL/9=4{,}067{,}000$, the least possible value of the constant applied bending moment per span. The distribution of the loading was not uniform and was such as would probably increase the applied moment WL somewhat.

It is not possible to determine from the test data exactly what the average resistance of the steel in the belts was, but outside figures can be readily assigned. The mean value of the unit stress at three gage lines at mid-span in a side belt parallel to the wall was nearly 9400 lb., and at mid-span of the diagonal belt was 14,600 lb.

Take $d_1=7.3$ in., $d_2=6.95$ in., $d_3=6.5$ in.

Then the moment of resistance of the steel at mid-span due to all 30 rods of the two side belts was

$$0.91\times7.3\times9400\times30\times0.11=207{,}200 \text{ in.-lb.,}$$

and the moment of resistance of the steel in the diagonals at mid-span was

$$0.89\times6.95\times14{,}600\times15\times0.11=153{,}100 \text{ in.-lb.,}$$

making a total at mid-span of 360,300 in.-lb.

Somewhat greater uncertainty exists respecting the mean unit stresses in the belts across the top of the columns than elsewhere, since the only available observations were made upon gage lines on diagonal belts over the columns which show mean unit stresses of 10,200 lb. If this be assumed

* See *Trans. Am. Soc. C. E.*, Vol. LXXVII (1914), p. 1383, and "Concrete-Steel Construction," Eddy and Turner, p. 277.

as the unit stress in all the belts, direct as well as diagonal, then one-half the moment of resistance of the steel over both columns at one end of the span, reckoning all the rods in the two side belts and two diagonal belts as efficient, was

$$0.83 \times 6.5 \times 10{,}200 \times 30 \times 0.11 = 186{,}500 \text{ in.-lb.}$$

or the sum total of half the resisting moment at the two ends, or twice this, was 373,000 in.-lb., which, together with the moment at mid-span, made 733,300 in.-lb. The constant applied moment per span $WL/9$ is 5.55 times this resisting moment.

It will be noticed in this case of a single loaded panel that the numerical value of the moment at mid-span is practically the same as each end moment.

If the constant applied bending moment per span be taken at $WL/8 = 457{,}500$ instead of at $WL/9$, by reason of the additional load of $W/8$ or thereabouts, due to the vertical shear at the edges of the strip assumed to have a width of $2L$, as was assumed in the Larkin test, an assumption, which would probably be more nearly correct than the one made above, it appears that $WL/8$ is 6.2 times the computed resisting moment of the steel, which may be taken to show the effect of the neighboring wall in this test, as was done in the Larkin test.

Load 3 was one in which $W = 95{,}000$ lb. per panel on each of four panels forming a square extending for a distance of two panels along the outside wall.

Taking $L = 200$ in., $WL = 18{,}000{,}000$ in.-lb., then $WL/9 = 2{,}000{,}000$ in.-lb. may be taken as a minimum value of the constant applied bending moment per span. But the load which was not uniformly distributed over the area of the panels is thought to have caused bending moments equivalent to a somewhat greater load than this if uniformly distributed.

Consider one span of a beam strip parallel to the wall across the middle of the loaded area having a width of a single panel with a row of columns along its center line, so that one-fourth of the entire loaded area lies on each side of this strip. Then the moment of resistance of the steel at mid-span of the side belt extending along the middle line of the strip would be

$$0.91 \times 7.3 \times 7700 \times 15 \times 0.11 = 86{,}800 \text{ in.-lb.,}$$

and the moment of resistance at mid-span of the diagonal belt was

$$0.89 \times 6.95 \times 6400 \times 15 \times 0.11 = 67{,}100 \text{ in.-lb.,}$$

making a total at mid-span of the strip of 153,900 in.-lb.

The gage lines observed at both ends of the span were all on diagonal rods, and we shall assume as an outside figure that the unit stresses in the rods of the side belts over the columns are equal to those in the diagonals, as they cannot apparently exceed that figure. The average unit stress in the diagonal steel over the center column of the strip was 15,900 lb. and over the end column of the strip was 5320 lb. Consequently one-half the sum of the resisting moments of the steel over these two columns, which are at the two ends of the span in question, was

$$0.83 \times 6.5\ (15{,}900 + 5320)\ 15 \times 0.11 = 193{,}900 \text{ in.-lb.}$$

making the sum total of half the resisting moments at the ends plus that at mid-span = 347,800 in.-lb. The numerical value of the applied bending moment per span, $WL/9$, is 5.75 times this resisting moment of the steel, which shows that it receives considerable assistance from the wall, etc. If the constant applied moment be taken in this case to be $WL/8$, as no doubt is more nearly correct, that will be a somewhat larger multiple of the steel resistance per span than the number just calculated.

It will be noticed that with this distribution of loading the resisting moments at that end of the span which lies in the middle of the strip are almost exactly three times as great as at the other end of the span, a fact which corresponds closely to the case of the strip treated in the Larkin test.

The other test loads that were placed upon this floor were too unsymmetrical to be satisfactorily treated by this method.

11. Test of the St. Paul Bread Company Building, by Prof. W. H. Kavanaugh.*

Thickness of rough slab, 6 in.; size of panels 15 x 16 ft.; design load, 100 lb. per sq. ft., which was subjected to a total maximum test load, W of 100,000 lb. on a single panel, or 416.8 lb. per sq. ft.; L_1, 192 in. Hence $WL_1 = 19{,}200{,}000$ in.-lb., and $WL_1/8 = 2{,}400{,}000$ in.-lb. Each belt consisted of 10 $\frac{3}{8}$-in. round rods.

Consider a strip the long way of the panel, two panels wide, to include the side belts of the loaded panel. Then the moment of resistance of the two side belts of 20 rods at mid-span must be somewhat less than that obtained from the unit stresses observed in gage lines all of which lie under the edges of the loaded area, or less than

$$0.91 \times 5.3 \times 17{,}200 \times 20 \times 0.11 = 183{,}400 \text{ in.-lb.}$$

The moment of the steel in the diagonals at mid-span was

$$0.89 \times 5 \times 6000 \times 10 \times 0.11 = 29{,}600 \text{ in.-lb.},$$

making a total resisting moment at mid-span = 213,000 in.-lb.

Now assume in default of observations that the moment at each end is equal to that at mid-span, which must be nearly the fact, as it was in the Northwestern Glass Company Building, when a single panel only was loaded, and the sum total of one-half the end moments plus that at mid-span = 426,000 in.-lb. Then the numerical value of $WL_1/8$ is 5.63 times this sum, a result which agrees with the tests previously discussed.

It thus appears that the theory of steel resistance in flat slabs which was proposed at the beginning of this paper for flat slabs of this type is confirmed by the whole body of test data at our disposal, and the fact of a practically constant relation between the applied and resisting moments per span must be admitted, without regard to whether the explanation which was suggested be accepted as correct or rejected as incorrect.

12. Comparison of constant applied bending moment per span with flat-slab theory.†

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* See *Trans. Am. Soc. C. E.*, Vol. LXXVII (1914), p. 1376, and "Concrete-Steel Construction," Eddy and Turner, pp. 185 and 232.

† See "Concrete-Steel Construction," Eddy and Turner, Chapter V.

According to equation (34a), page 184, of the book referred to in the foot-note, the resisting moment of the steel at mid-span of a side belt is $WL/192$, and by equation (52), page 196, that at mid-span of the diagonal belts is $WL/256$, making the sum total of resisting moments at mid-section of the panel about $WL/106$, or $WL/53$ in 180 deg. about a column. Assuming the most unfavorable case for steel stresses at mid-span, viz.: a single panel only loaded, where, as we have seen, the resisting moment at mid-span is practically equal to that at each end, the applied moment at mid-span is $\frac{1}{2}WL/9 = WL/18$. This is almost six times the resisting moment as just determined from slab theory, on the basis of all spans uniformly and equally loaded, and is consequently somewhat less than six times the resisting moment for the case of a single loaded panel.

Since the case of one panel only loaded is the one most unfavorable to the steel across the section at mid-span, and the moment of resistance at mid-span cannot exceed the half sum of the moments of resistance of the steel at the ends of the span, it is clear that a conservative estimate of the greatest moment of resistance of steel at mid-span cannot exceed one-fifth of one-half the applied bending moment per span, or one-tenth of $WL/8 = WL/80$, or $WL/40$ for all the steel at mid-span in 180 deg. around a column. This, at an assumed working unit stress of 16,000 lb. for mild steel, is the same as $WL/50$ for an assumed working unit stress of 13,000 lb., which last expression is the formula for design of flat slabs published by C. A. P. Turner in 1909 in his treatise on "Concrete-Steel Construction," a result which he had at that time already deduced from experimental data at his disposal at a date long prior to the advent of extensometer measurements of steel stresses. It appears that this early deduction of his has been amply confirmed by these later tests made with all the care that can be exercised by trained experimentalists equipped with instruments embodying every refinement that mechanical science has been able to suggest.

This formula, however, instead of having been generally accepted as valid and recognized as correct, has been attacked most unsparingly as expressing relations utterly impossible of realization, and as attempting to furnish the basis for designs with percentages of reinforcing steel so small as to make buildings erected in accordance therewith wholly unsafe. And nevertheless it appears that this formula is the conservative expression of relations with which the really up-to-date and most authoritative tests that have been published on flat slabs agree, tests whose fundamental accuracy is beyond question since they were made by trained observers who have staked their reputation upon them.

Again, a conservative estimate of the resisting moment of the steel required across the section between column centers may be made as follows:

Since the greatest applied moment occurs at this section when the panels are all loaded, that is the most unfavorable arrangement for the steel at this section. The maximum applied end moment in this case is $\frac{2}{3}WL/8$ with no reduction by reason of the size of the cap. The resisting moment is less than one-fifth of this, or $WL/60$, i. e., $WL/30$ in 180 deg. around a column center. This consequently is a conservative estimate of the resisting moment

of the steel at the edge of the cap, and is only one-half of that previously mentioned as common practice.

If one-half of the rods forming the belts at mid-span have long laps over the column heads so as to increase their moment of resistance over the columns by 50 per cent, that will increase the moment previously determined at mid-span, viz., $WL/40$, to about $WL/27$ at supports, which will furnish ample resisting moment over the columns.

This conservative estimate makes the steel sufficient to furnish a sum total of resisting moments per span amounting to

$$WL/60+WL/80=7WL/240.$$

This is so large an amount of steel that the largest possible applied bending moment per span, which is $WL/8$, is only 4.3 times the resisting moment instead of $5\frac{1}{3}$ times as much, thus showing a large margin of excess section of steel over that required for uniform load on all panels at once.

The reason why the applied moment in this case is so small a multiple of the resisting moment of the estimated cross-section of the steel as 4.3 times it, is evident when we consider that the mid-section is estimated to resist the maximum stress for a load on a single panel only, while the estimated cross-section of steel at the end section is not for this same loading, but is estimated instead to resist a maximum stress occurring when all the panels are uniformly and equally loaded. These two conditions do not co-exist at one and the same time, and the steel so estimated as to cover both requirements must exceed that required by either condition separately, which last is the case to which the theorem applies.

13. The following is a comparison of the results that have been arrived at by the foregoing theory and strikingly confirmed by the tests quoted in this paper with the new ruling on the design of reinforced concrete flat slabs put in force in the city of Chicago,* which is explicitly put forth as based upon an accumulation of test data in the office of the Commissioner of Buildings of the city of Chicago.

Translated into the notation employed in this paper, the ruling in effect requires in a two-way system of reinforcement that the positive resisting moment of the steel at mid-span of a side belt and for a width of section of $L/2$ shall be $WL/60$, and for the remaining width of $L/2$ between side belts shall be $WL/120$, and further that the negative resisting moment for an equal width over a column shall be $WL/30$, and for the remaining width of $L/2$ between columns shall be $WL/120$. This requires for this system that the sum total of half the resisting moments at the ends of the span plus that at mid-span shall amount to $WL/15$. Similarly in case of a system of four-way reinforcement this ruling requires a total resisting moment per span of $WL/16$.

It appears, however, from the preceding conservative estimate, that in neither system is it required that this sum total shall exceed $7WL/240 = WL/34.3$. The ruling, therefore, in each case demands more than twice the

* For text of ruling and comment see *Engineering News*, September 24, 1914.

the steel that is necessary, as shown by all available experimental data obtained by observations upon flat slabs of this type of reinforcement.

In view of the foregoing concurrent and cumulative evidence on this question, the writer is convinced that it must be a misstatement of facts to say that there is any accumulated mass of experimental data in the office of the Commissioner of Buildings in Chicago or elsewhere that will support any such requirements as those contained in the Chicago ruling.

There has for years been a radical divergence of opinion among engineers as to the amount of reinforcement required in flat slabs. Those who have relied upon beam theory alone have concluded that about twice as much steel is needed* as those having practical acquaintance with the strength of slabs know is actually required. But this is the first time, so far as known, that it has been asserted by any one that experimental evidence is in existence to support any such contention.

14. As previously stated, the theorem which has been developed and applied in this paper respecting the constant value of half the sum of the end moments plus the moment at mid-span of a panel has to do with the load in any given single panel which is under consideration and with no other panel load. But the question of the relative magnitude of the maximum moments and stresses in the several panels, although it is of great importance in slab design, is one on which no light has been shed thus far by this discussion. It therefore now claims our attention, and more particularly with reference to the relative stresses in wall panels as compared with interior panels.

Since it is a fact that the moments near the middle of end spans of continuous beams of many spans under uniform loads with free ends are about twice those in other spans,† it has been too generally assumed that the same ratio also exists in floor slabs supported on bearing walls without restraint along their free edges.

Such, however, is not the case since the mechanics of slabs is not the same as that of beams. It will appear later that the greatest stresses near mid-span of the side belts of wall panels exceed those in other panels by only about 50 per cent in case of slabs with free edges resting simply on bearing walls, while any restraint such as invariably occurs at the edges will make the excess of those stresses still smaller. In fact when the restraint at the edges of the slab is such as to furnish a resistance to flexure practically equal to that afforded in the interior of the slab at the supporting columns, the stresses in wall panels will not differ materially from those in other panels. Such efficient restraint may be afforded by a row of columns with heads standing just in front of the face of the wall together with a side belt over it parallel to the wall, etc. Restraint of various intermediate degrees of efficiency may be afforded by stiff elbow rods in wall pilasters and slab rods most of which run well into the wall at the top of the slab so as to produce cantilever action at the wall and prevent any cracks in the top of the slab next the wall.

All such arrangements are intermediate between the two ideal limiting cases of perfect restraint and zero restraint at the walls. In the case of per-

* See *Trans. Am. Soc. C. E.*, Vol. LXXVII (1914), p. 1448.

† See "Concrete-Steel Construction," Eddy and Turner, p. 100, Fig. 39.

fect restraint at the edges the stresses in the wall panels are practically the same as in the other panels, while with free edges the stresses at mid-span will not exceed those at mid-span in other panels by more than about 50 per cent, and the stresses in cases intermediate between these limits may be closely estimated for any given structure according to the circumstances of the case.

The stresses in the limiting case of the wall panels of a floor slab where its edges rest freely without restraint on bearing walls are to be found approximately by help of the lines of inflection, as follows:

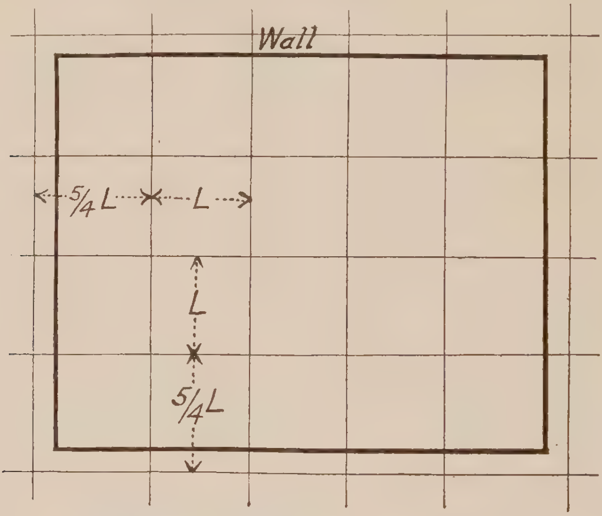

FIG. 1.—PLAN OF THE PANELS OF A FLOOR SLAB UNIFORMLY LOADED RESTING ON BEARING WALLS, AND OF THE PANELS OF AN INDEFINITE EXTENSION OF THE SAME WITHOUT WALLS AND UNIFORMLY LOADED, IN WHICH THE STRESSES IN THE EXTENDED WALL PANELS ARE PRACTICALLY THE SAME AS THOSE IN THE ORIGINAL WALL PANELS.

In this case one line of inflection of the wall panel is at the wall and the other may be taken with sufficient accuracy at a distance of $L/4$ from the column centers. That part of the wall panels lying between these lines of inflection and forming a strip along the wall of width $3L/4$ is a valley parallel to the wall with a strip of width $L/4$ along the side of it next to the first row of columns which forms one-half of the ridge over the columns of that row. Now suppose that without any changes of curvature in this part of the slab the wall be removed and an indefinite extension of the slab be made at that edge with a row of columns placed at a distance of $L/4$ back of the line of the wall with a strip forming a ridge over them like that over the first row. If this were to occur all around the floor, then each wall panel might be regarded as forming a part, viz.: four-fifths, of a completed panel of a size $L \times 5L/4$.

This is represented by the panels in Fig. 1, in which the interior panels (of span L) are square, the wall panels are $L \times 5L/4$, and all others $5L/4 \times 5L/4$. Such a floor of an indefinitely large number of panels, each with a uniformly distributed load of w lb. per sq. ft., would be one in which the stresses in the interior and wall panels would be practically the same as those in the slab considered. This, however, would not be true with respect to the corner panels where the sharp diagonal valleys due to the horizontal wall supports in two directions will require steel across the corners to resist local stresses. The mechanics of such valleys is discussed on page 286 of "Concrete Steel Construction," previously mentioned.

The stresses at mid-span in the side belts of the completed wall panels arising from a load of w lb. per sq. ft., or a total load of $W = 5wL^2/4$, are $5/4 \times 5/4 = 1.56$ times as much as in the side belts of interior panels in case of no restraint whatever at the edges except mere support, a case which can hardly occur in practice except where the slab rods run into bearing walls at the bottom of the slab and the concrete cracks along the wall at the top of the slab. Were it not for the local stresses in the corner panels previously mentioned the stresses in the side belts of corner panels would be those due to a uniform load of w lb. per sq. ft. in a panel of size $5L/4 \times 5L/4$, which would be 5/4 times that in an ordinary wall panel by reason of the greater total load, or $1.56 \times 5/4 = 1.95$ times that in an interior panel. This will be reduced somewhat by the local reinforcement previously mentioned, so that the belts in corner panels of slabs with free edges will never be subjected to stresses that are more than twice as great as those in interior panels, while in ordinary construction they are much less than that.

15. Conclusion.—Theory and test results show that ordinary requirements for slab design which are based upon resistance to applied moments according to beam theory, prescribe twice as great cross-section of steel in the slab rods as is needed to develop the compressive strength of the concrete.

TIME TESTS OF CONCRETE.

By Almon H. Fuller* and Charles C. More.†

In making a recent test (April, 1915) of a reinforced concrete building the writers found concrete deformations in the building which greatly exceeded those in the laboratory tests, under, presumably, approximately the same

Table I.—Concrete Specimen A.

Deformations in Dial Units in 8 in. for Days and Loads indicated.
1 Dial Unit = 0.0002 in.

400 lb. per sq. in.		650 lb. per sq. in.		900 lb. per sq. in.		1150 lb. per sq. in.		No load.	
Days.	Def.	Days.	Def.	Days.	Def.	Days.	Def.	Days.	Def.
2	7.1	3	11.4	4	15.3	6	22.0	13	29.9
3	8.3	4	12.5	5	17.3	7	26.7	14	26.9
..	....	..	....	6	19.0	..	....	15	25.7
..	....	..	....	..	....	1300 lb.	per sq. in.	16	25.6
..	....	..	....	..	....	7	28.4	18	26.9
..	....	..	....	..	....	8	32.4	21	27.4
..	....	..	....	..	....	9	34.6	..	....
..	....	..	....	..	....	1450 lb.	per sq. in.	..	....
..	....	..	....	..	....	9	36.7	..	....
..	....	..	....	..	....	10	40.1	..	....
..	....	..	....	..	....	11	43.7	..	....
..	....	..	....	..	....	12	46.2	..	....
..	....	..	....	..	....	13	48.0	..	....
..	....	24	36.8	..	....	..	....	..	....
..	....	25	38.5	..	....	..	....	..	....
..	....	26	38.6	..	....	..	....	..	....
..	....	31	41.8	..	....	..	....	..	....
..	....	34	43.1	..	....	..	....	..	....
..	....	38	45.9	..	....	..	....	..	....
..	....	41	46.6	..	....	..	....	..	....
..	....	44	47.2	..	....	..	....	..	....
..	....	48	49.2	..	....	..	....	..	....
..	....	55	52.2	..	....	..	....	..	....
..	....	60	53.6	..	....	..	....	..	....
..	....	65	54.9	..	....	..	....	..	....
..	....	70	57.2	..	....	..	....	..	....
..	....	79	58.3	..	....	..	....	..	....

Concrete Specimen A, size 8 in. diameter by 15 in. high; cast from roof concrete on January 13, 1915; testing begun April 24, 1915; age 101 days.

unit stresses. It was found, further, that the deformations in the building increased during the few days that the maximum test load was allowed to remain. It seemed probable, therefore, that the time element should be

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considered in interpreting the stresses, but a search in available literature failed to disclose any data on the subject.*

Use was then made of three available compression specimens in applying increments of loads at intervals of 24 hours at approximately the same rate as in the building.

The results are given in Tables I, II and III, and are shown graphically on Figs. 1 to 5. It will be noted that all specimens were loaded in the same manner for 6 days and each treated separately after that time. Other needs

TABLE II.—CONCRETE SPECIMEN B.

Deformations in Dial Units in 8 in. for Days and Loads Indicated.
1 Dial Unit = 0.0002 in.

400 lb. per sq. in.		650 lb. per sq. in.		900 lb. per sq. in.		1150 lb. per sq. in.		
Days.	Def.	Days.	Def.	Days.	Def.	Days.	Def.	Corrected Def.
2	8.3	3	12.1	4	15.6	6	21.9	....
3	9.4	4	13.1	5	17.6	7	25.8	....
..	....	..	....	6	19.2	8	27.7	....
..	....	..	....	..	....	10	30.8	....
..	....	..	....	..	....	12	33.6	....
..	....	..	....	..	....	14	36.3	....
..	....	..	....	..	....	18	41.4	....
..	....	..	....	..	....	21	43.4	....
..	....	..	....	..	....	26	47.6	....
..	....	..	....	..	....	31	52.7	....
..	....	..	....	..	....	38	59.5	....
..	....	..	....	..	....	48	64.9	....
..	....	..	....	..	....	55	70.0	....
..	....	..	....	..	....	60	73.1	....
..	....	..	....	..	....	70	79.6	....
..	....	..	....	..	....	79	82.0	....
..	....	..	....	..	....	87	84.5	....
..	....	..	....	..	....	93	87.2	....
..	....	..	....	..	....	103	88.8	....
..	....	..	....	..	....	122	94.7	93.3
..	....	..	....	..	....	145	101.1	97.5
..	....	..	....	..	....	166	103.2	100.8
..	....	..	....	..	....	210	112.7	108.0
..	....	..	....	..	....	228	116.7	110.7
..	....	..	....	..	....	259	122.4	114.9
..	....	..	....	..	....	266	122.8	115.6
..	....	..	....	..	....	273	124.8	116.8
..	....	..	....	..	....	280	124.5	117.2

Concrete Specimen B, size 8 in. diameter by 12 in. high; cast from second floor concrete on December 8, 1914; testing begun April 24, 1915; age 137 days.

for the testing machines influenced this treatment to some extent. Figs. 4 and 5 show the relation between days under load and deformation during the periods that the specimens were under constant load.

It seems apparent that the modulus of elasticity as a measure of the relation of stress to deformation while the load is being applied or released

* Since this time the paper by Prof. F. R. McMillan, of the University of Minnesota, giving results of time tests of a number of reinforced concrete beams and slabs, has become available. This is published by the University of Minnesota.

is of doubtful value in determining the stresses that are represented by observed deformations in concrete that has been under load for a considerable period of time. However, the modulus of elasticity as a measure of the relation of stress to deformation as determined from time tests in which the rate of application is the same would furnish means for making a fairly satisfactory determination of the existing stresses.

In Figs. 1, 2 and 3 an average line has been drawn through the points showing the deformations 24 hours after each load was applied, and the modulus of elasticity corresponding to each of these lines has been noted.

TABLE III.—CONCRETE SPECIMEN C.

Deformations in Dial Units in 8 in. for Days and Loads Indicated.
1 Dial Unit = 0.0002 in.

400 lb. per sq. in.		650 lb. per sq. in.		900 lb. per sq. in.		1150 lb. per sq. in.		No load.	
Days.	Def.	Days.	Def.	Days.	Def.	Days.	Def.	Days.	Def.
..	4.6	1	8.2	2	11.9	4	17.6	7	11.1
1	5.6	2	9.5	3	13.1	5	21.1	8	9.9
..	....	..	....	4	15.1	6	23.2	9	10.1
..	....	..	....	..	....	7	24.7	10	10.0
..	....	11	15.0	..	....	..	....	11	10.4
..	....	12	18.2	..	....	..	....	..	....
..	....	14	21.5	..	....	..	....	..	....
..	....	16	23.7	..	....	..	....	..	....
..	....	19	24.3	..	....	..	....	..	....
..	....	21	26.1	..	....	..	....	..	....
..	....	24	27.3	..	....	..	....	..	....
..	....	29	30.3	..	....	..	....	..	....
..	....	32	31.7	..	....	..	....	..	....
..	....	36	34.0	..	....	..	....	..	....
..	....	39	34.9	..	....	..	....	..	....
..	....	42	34.8	..	....	..	....	..	....
..	....	46	37.0	..	....	..	....	..	....
..	....	53	39.6	..	....	..	....	..	....
..	....	58	41.3	..	....	..	....	..	....
..	....	63	42.1	..	....	..	....	..	....

Concrete Specimen C, size 8 in. by 8 in. by 13 in. high; cut from third floor, which was cast on December 16, 1914; testing begun April 26, 1915; age 131 days.

These moduli will serve as a basis for determining stresses within corresponding deformations and days. They will doubtless give results in excess of actual stresses for deformations greater than those observed.

Tables I to III will be found helpful in such cases. For instance, a deformation of 25 dial units in 7 days would represent a stress in the neighborhood of 1150 lb. per sq. in.; that is, the 7-days deformations for 1150 lb. for the three specimens are 26.7, 25.8 and 24.7 respectively. Likewise the one available specimen in each case would indicate a stress of 1450 lb. per sq. in. for 48.0 at 13 days and 1150 lb. per sq. in. for 94.7 at 122 days.

In presenting these few results at this time it is realized that they are entirely inadequate for use other than on the building in question. It is

believed, however, they indicate that the plasticity of concrete must be recognized and the distribution of stress between steel and concrete in reinforced concrete structures may be quite different from that usually assumed. It is hoped the discussions may bring out the results of other work done along this line.

The standard to which the readings have been referred is a steel bar bedded in concrete; therefore the results show the total deformations in the concrete, including shrinkage, effect of humidity, etc.

As there was no thought at the beginning of continuing the readings more than a week or two, no effort was made to observe deformations other than for stress.

The readings beyond the 103d day after the load had been released from cylinders A and C show, by a comparison of B with A and C the deformation from stress alone. This has been entered in Table II and plotted upon Fig. 5 by a curve marked "Corrected B."

The writers wish to acknowledge their indebtedness to their colleague, C. C. May, Instructor in Civil Engineering at the University of Washington, for taking a number of readings and for other assistance.

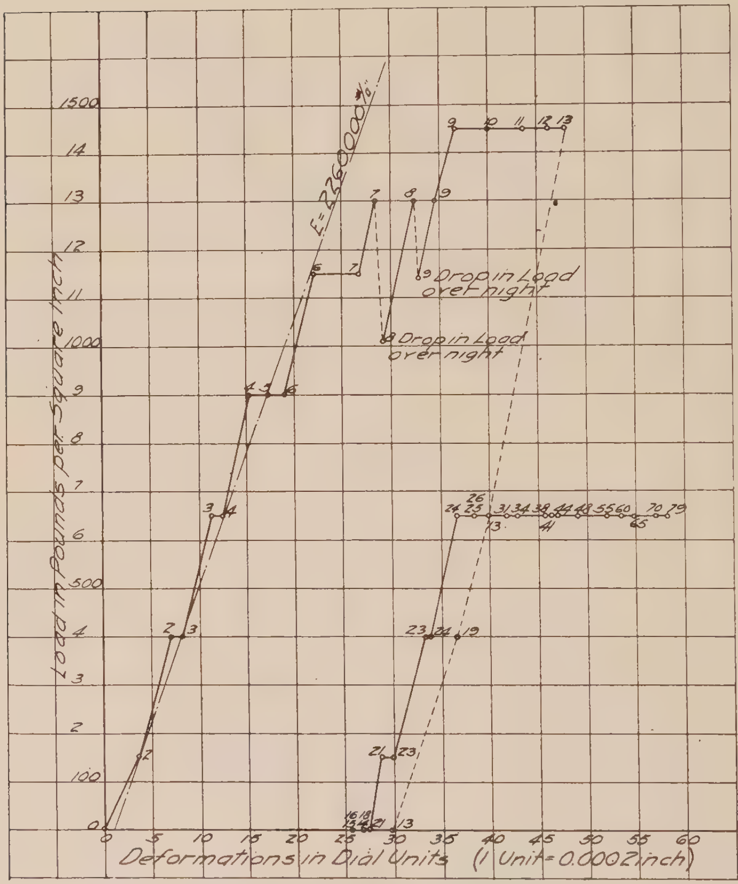

FIG. 1.—CONCRETE SPECIMEN A.

NOTE.—Specimen cast when roof was poured 101 days old when first load was applied. Small figures denote days under load.

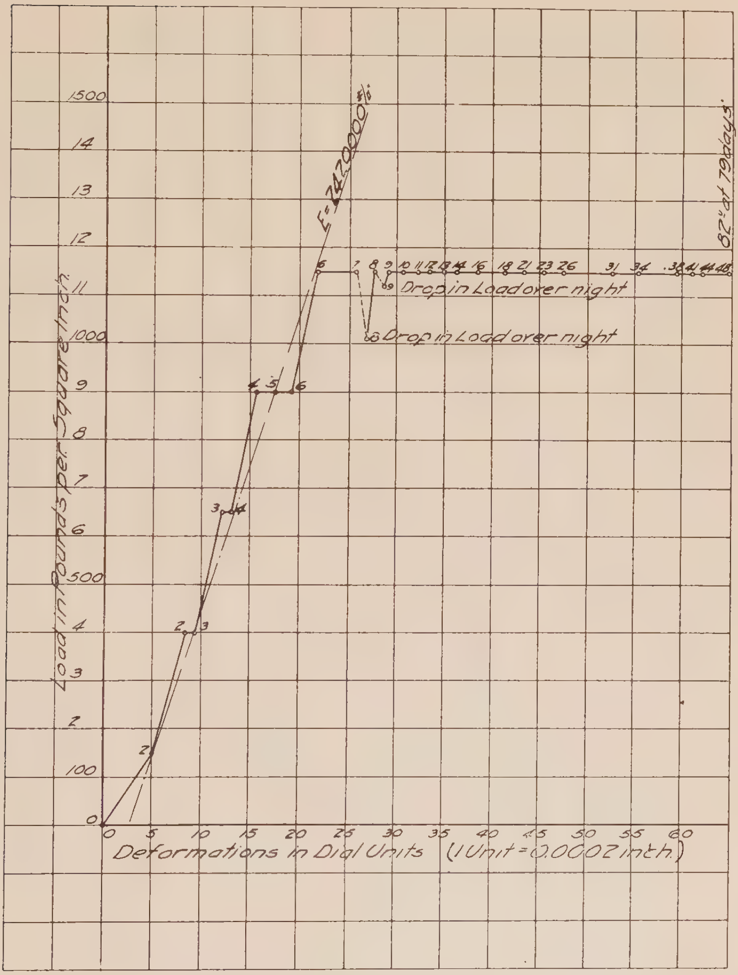

FIG. 2.—CONCRETE SPECIMEN B.

NOTE.—Specimen cast when second floor was poured 137 days old when first load was applied. Small figures denote days under load.

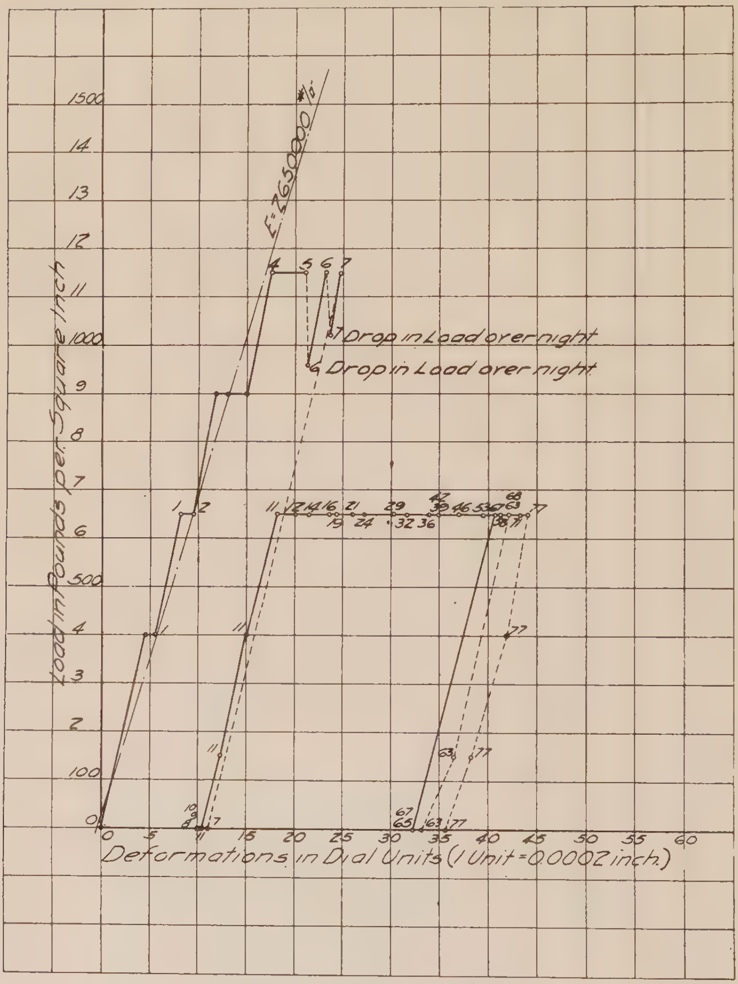

FIG. 3.—CONCRETE SPECIMEN C.

NOTE.—Specimen cast from third floor 4–26–15, 131 days old when first load was applied. Small figures denote days under load.

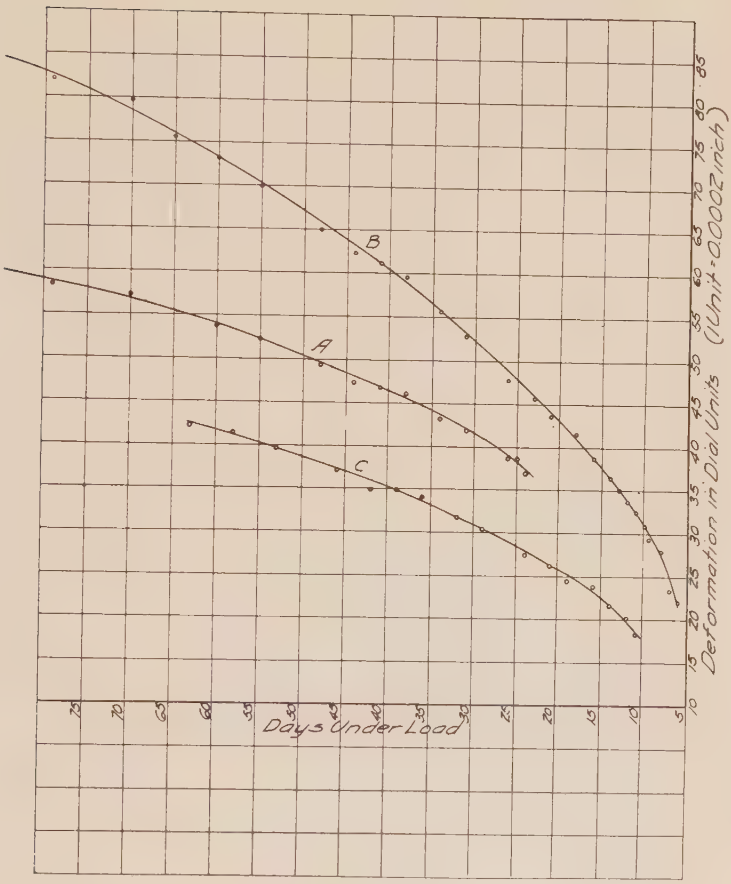

FIG. 4.—DEFORMATION OF CONCRETE UNDER CONSTANT LOAD.

NOTE.—Load for A = 650 lb. per sq. in. For B = 1150 lb. per sq. in. For C = 650 lb. per sq. in. See Figs. 1, 2, 3 for deformations from other loads.

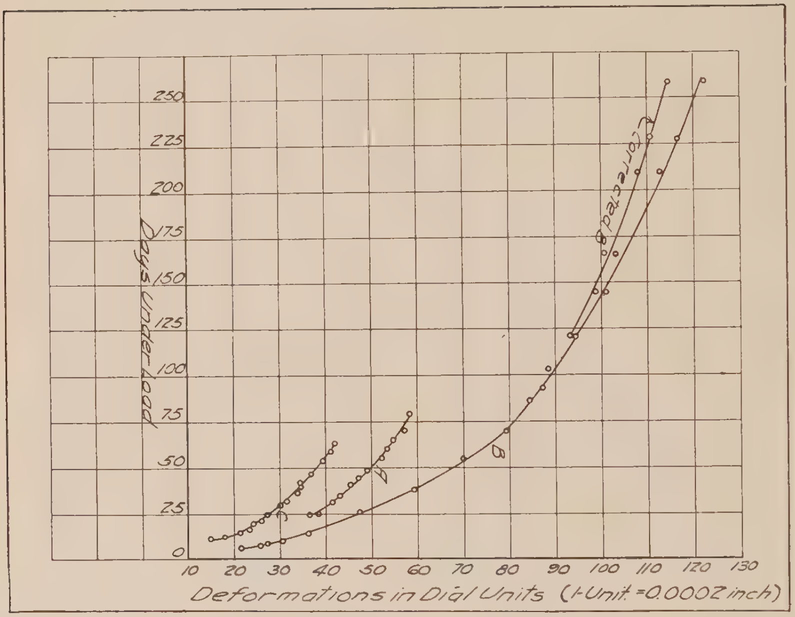

FIG. 5.—DEFORMATION OF CONCRETE UNDER CONSTANT LOAD.

NOTE.—Load for A=650 lb. per sq. in. For B=1150 lb. per sq. in. For C=650 lb. per sq. in. See Figs. 1, 2, 3 for deformations from other loads.

DISCUSSION.

Prof. McMillan.*—The authors of this paper were directed to this investigation by a circumstance similar to that which brought this subject to the speaker's attention in 1912. In applying a load to a slab for an acceptance test, a delay in the deliveries of the loading material brought out the fact that during an interval of nine days with two-thirds of the design load, the deflection increased from 0.18 to 0.29 in., and after further loading and delay the deflection of the slab under a load only 8 per cent in excess of its designed load increased from 0.41 to 0.58 in. in one week. Not long after this a series of investigations was begun which has been continued as the facilities and means permitted, until the present time considerably more data are at hand. Some of these have been published in a Bulletin of the University of Minnesota,† and when certain of the more important investigations under way are sufficiently advanced to warrant it a further publication may be looked for. Some of the data are presented here to illustrate the discussion which follows. Prof. McMillan.

There are two conclusions to be drawn from the data presented by these authors, and confirmed by his own experiments, that the speaker particularly desires to emphasize. The first of these is that the rate of time yield not only increases with the load, but increases in a somewhat higher ratio. That this is true can be seen from the curves of Fig. 4 of the paper under discussion, in which the curve for cylinder B, loaded to 1150 lb. per sq. in., is seen to rise more rapidly than that for either cylinder A or C which were held under the load of 650 lb. per sq. in. That the curve for cylinder A is higher than that for C, although practically parallel to it, is due to the difference in treatment during the early loading, as can be seen by reference to Figs. 1 and 3. The speaker believes that this increasing rate of yield with load is an important item and points to the desirability of using lower compressive stresses in concrete, at least in concrete such as we are now making. Since for a given yield at the extreme fiber, the amount of angular change between sections defining a short length of beam varies inversely as the depth of slab, it will be seen that for thin slabs or shallow beams and long spans the sag due to this yielding may become quite pronounced. The noticeable sags frequently observed in floors of flat slab construction are undoubtedly, to a large extent, due to the high rate of yielding that accompanies the abnormal compressive stresses known to exist adjacent to the column heads.

The second conclusion to be drawn from the tests recorded in this paper relates to a method of determining with considerable accuracy the compressive stresses in the concrete of a building under test. The authors' statement

* Assistant Professor of Structural Engineering, University of Minnesota.

† Studies in Engineering, Bulletin No. 3, "Shrinkage and Time Effects in Reinforced Concrete."

Prof. McMillan.

that "the modulus of elasticity as a measure of the relation of stress to deformation as determined from time tests in which the rate of application is the same would furnish means for making a fairly satisfactory determina-

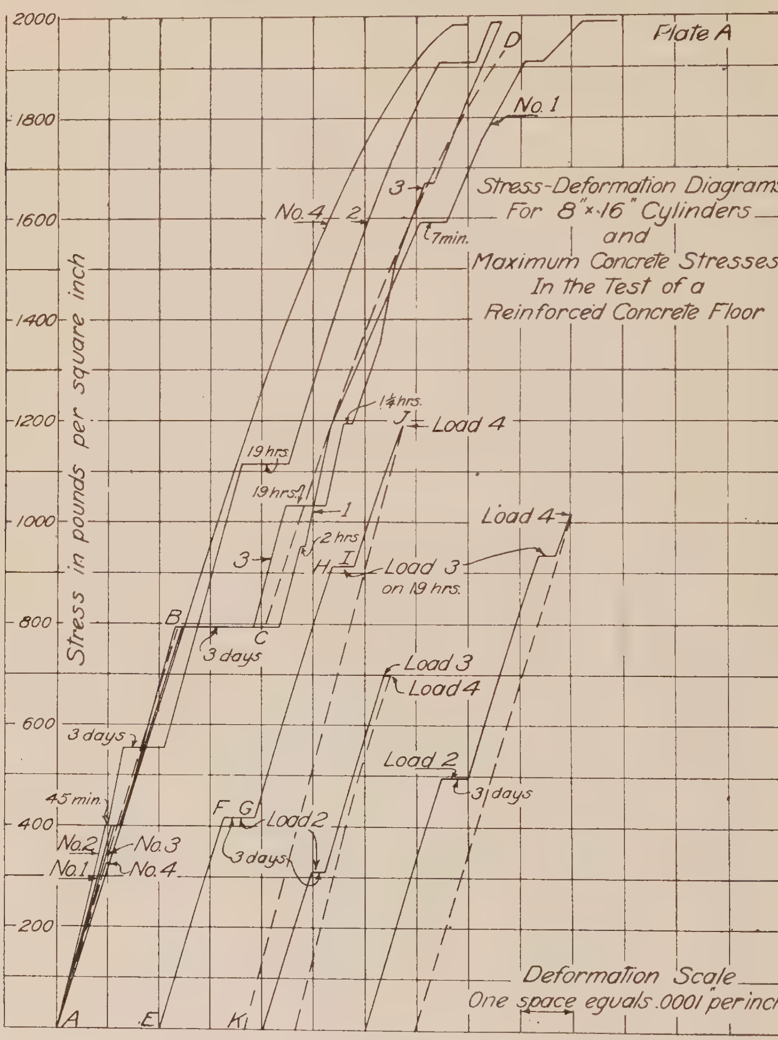

FIG. 6.

tion of the existing stresses," is in thorough accord with the speaker's deduction from his own investigations. The method of applying this principle to the test of a building slab is illustrated by the curves of Fig. 6.

Here are shown the stress deformation curves of four 8 x 16 in. cylinder specimens taken at the time the panel under test was cast. The curves 1, 2, 3 and 4 were obtained for different rates of loading. In two cases, Nos. 2 and 3, the periods for which the successive loads were held were identical with those for which the corresponding test loads remained on the slab. One cylinder, No. 4, was tested to failure without holding the load at any time longer than the usual interval required for the scale beam to maintain a balance for a number of seconds. The curves are practically parallel at all stages of loading and the dotted curve, A B C D, offset at the load of 800 lb. per sq. in. from B to C for convenience in averaging, represents quite closely the average slope of the curves at all points of its length. **Prof. McMillan.**

To interpret the stresses in the slab from a set of deformations for a particular gage line, a line is drawn parallel to the average curve until a deformation is reached corresponding to the deformation shown by that gage line for one of the test loads, and the corresponding stress read from the scale of ordinates. From here the line is drawn horizontally a distance representing the yield in that gage line during the interval between the finishing of one load and the beginning of another, and then projected parallel again to the average cylinder curve until a deformation is reached equaling that found for the next load on the test panel. As before, the stress is read from the scale of ordinates. Thus the curve composed of the segments EF, FG, GH, HI and IJ was drawn showing slab stresses of 420, 910 and 1390 lb. per sq. in. respectively for loads 2, 3 and 4 of the test. Similarly the other curves are drawn for other gage lines. In these observations it is assumed that the cylinder specimens represent fairly well the same grade of concrete as exists in the slab. The possibility of a considerable difference in many cases, due to difference in curing, etc., should not be ignored.

The data presented by the authors in Figs. 1 to 3 indicates this same tendency to parallelism in the curves for release and repetition of loads. Obviously the rate of applying the several increments of load affects the direction of the curve between these successive stages or repetitions. In two of the curves of Fig. 6 not only was the interval under any load kept the same as in the floor test, but the time of bringing the cylinder to this new load was made approximately the same as that required to put the new floor load in place. Since this interval was usually small compared to the longer interval of rest, great refinement in this particular was found to be unnecessary.

The conclusion to be drawn from this similarity of the curves of the authors and those of Fig. 6 is that the results of a cylinder test carried without break from zero to failure could be used in interpreting the stresses by the method explained above. This is further confirmed by the curves of Fig. 7 which shows the parallelism of the several portions of the curve at different stages of loading and unloading. This figure was prepared from the test of a specimen to be used in interpreting the data from a slab tested under both long time and repeated loads.

The speaker does not wish to prolong this discussion unduly, but in view of the importance of this subject perhaps one or two typical illustrations of the effect of this time yield will not be out of place.

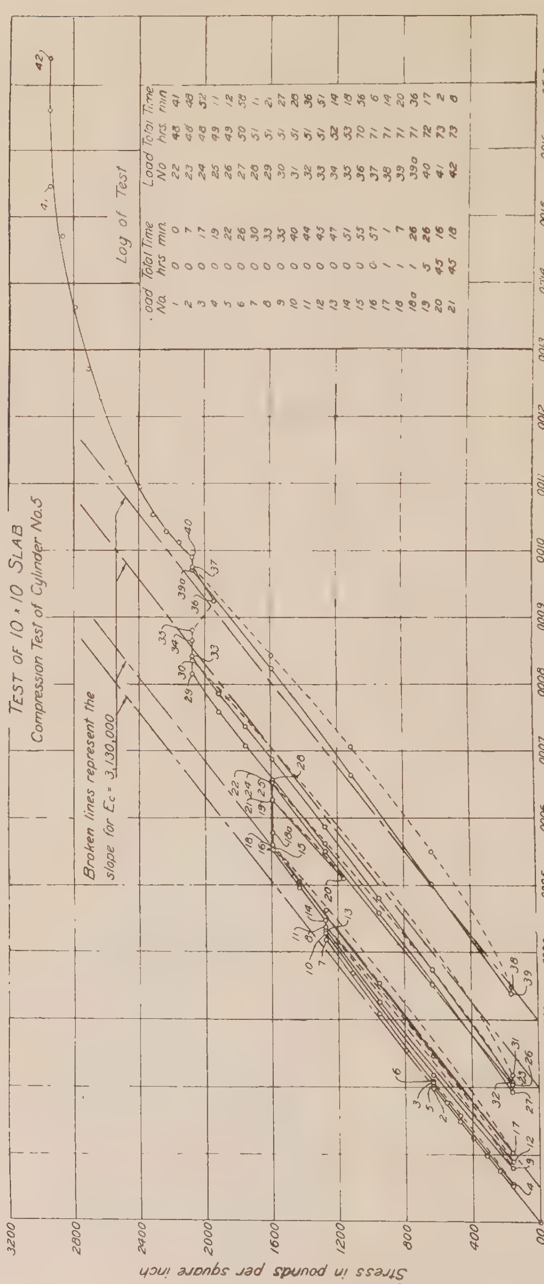
TEST OF 10 × 10 SLAB
Compression Test of Cylinder No. 5
Broken lines represent the slope for E_c = 3,130,000
Log of Test
Load No. | Total Time hrs. min. | Load No. | Total Time hrs. min.
1 0 0 | 22 48 41
2 0 7 | 23 48 48
3 0 17 | 24 48 52
4 0 19 | 25 49 11
5 0 22 | 26 49 12
6 0 26 | 27 50 38
7 0 30 | 28 51 11
8 0 33 | 29 51 21
9 0 35 | 30 51 27
10 0 40 | 31 51 28
11 0 44 | 32 51 36
12 0 45 | 33 51 51
13 0 47 | 34 52 14
14 0 51 | 35 53 18
15 0 53 | 36 70 56
16 0 57 | 37 71 6
17 1 1 | 38 71 14
18 1 7 | 39 71 20
180 1 26 | 390 71 36
19 5 26 | 40 72 17
20 45 16 | 41 73 2
21 45 18 | 42 73 8
Stress in pounds per square inch
Deformations in inches per inch

FIG. 7.

In Fig. 8 are shown the results of measurements made on several reinforced concrete columns in buildings in service. Columns marked 9, 19 and 20 are in one building in which the observations were begun about two months after casting and before all of the dead load from the floors above was in place. In fact, at the time of the last observations the building had just been completed. Columns 1 and 2 are from another building which was considerably more advanced when observations were begun, the full dead load or its equivalent in extra load during construction was probably in place at the time zero readings were taken. Although this building had been in service for six months when the last observations were made, no live load Prof. McMillan.

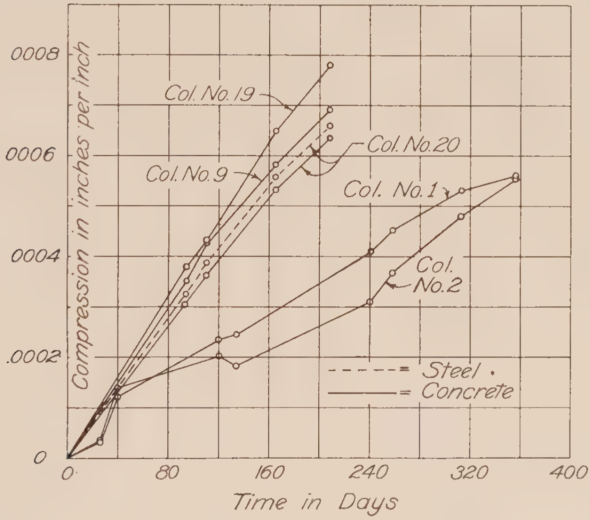

FIG. 8.—TIME DEFORMATION TESTS OF REINFORCED CONCRETE COLUMNS IN SERVICE.

need be considered, for it is a class-room building designed for loads far in excess of the average maximum condition.

In the columns of this building the effect of shrinkage is probably a small part of the total time movement shown, for with observations begun when the columns were several months old, the shrinkage had largely ceased.

In a study of these curves the magnitude of the deformation is the principal item of interest. In columns 1 and 2 the total compression of 0.00055 in. per in. corresponds to the compression produced by a stress of 1100 lb. per sq. in. (assuming the modulus of elasticity of the concrete equals 2,000,000), or a steel stress of 16,500 lb. per sq. in. if the rods have shortened a like amount. This condition, it should again be noted, is due only to the time yield under dead load and does not include the deformation produced

Prof. McMillan. when the dead load was first applied nor any of the live load for which the column was proportioned and but a fraction of the shortening due to shrinkage.

In the columns of the other building a larger part of the shrinkage is probably included, as well as some of the dead load increment applied as the building progressed. It will be noted here that at 210 days column No. 19 shows a movement equivalent to 22,000 lb. per sq. in. steel stress, and column 20, by actual measurement on one of the column verticals, shows a real stress of 19,200 lb. per sq. in.

About one year ago, in a discussion of the failure of the columns in the Edison fire,* the speaker stated that it was probable that the column verticals were under stresses approaching the elastic limit before the fire started. This statement was based on measurements from beams and slabs only. The results shown by the columns of Fig. 8 seem to the speaker to confirm entirely the probable accuracy of this statement.

In considering these curves of column deformations, the fact that the rate of increase has not noticeably reduced at 200 or 360 days, should be noted.

* See *Engineering News*, Vol. 73, page 502, March 11, 1915.

THE FLOW OF CONCRETE UNDER SUSTAINED LOAD.

By Earl B. Smith.*

The ordinary conception of the behavior of materials of construction under stress is that it follows Hooke's law for both increasing and decreasing loads within the limit of elasticity. As a matter of fact, this straight-line relation between stress and deformation is only approximately true for any material. There is an elastic-hysteresis phenomenon† in all cases of loading and unloading a material which makes the relation between stress and deformation only approximately constant. Although structural metals do not exactly follow this straight-line law, they so nearly do that stress computations based upon their deformations are approximately constant and independent of any time factor. It has long been realized that the deformations of stressed concrete are not even an approximately constant function of the stresses; but the effect of any time factor or of continually sustained loads has been almost entirely overlooked.

While making some tests on large reinforced concrete slabs in August, 1914, the writer noticed the large variation in the concrete deformations under a constant stress, and was unable to make any check readings after an elapsed time of one hour or more; even when the temperature and moisture conditions were constant. After considerable search for the cause of these variations, there was discovered an apparently definite relation between them and the elapsed time. Several experiments were made with the object of determining the amount of "flow" in the concrete, with respect to the time factor, under sustained loads.

A reinforced concrete beam was cast, and at the age of about two weeks was placed on two supports and loaded at the center with a single concentrated load of 1225 lb. The size of this beam was 5 in. wide, 8 in. full depth, and 7 in. effective depth; the supports were 10 ft. clear span; the reinforcement amounted to approximately 0.75 per cent, and consisted of five ¼-in. Havemeyer rods. The concrete was 1 : 2 : 4 gravel.

The strain-gage points were placed 8 in. apart on the top of the beam, which permitted readings to be made along its whole length, and thus gave the concrete deformations for different fiber-stress values. The beam when loaded had a computed fiber-stress of approximately 1000 lb. at the center section, 60 in. from the support; the fiber-stress at the 30-in. section would then be about 500 lb., and at other sections in proportion to their position.

The results of this test are shown graphically in Fig. 1. The flow of the concrete for four fiber-stress values (which are indicated only by the position of the strain-gage points, in inches from the support) is shown in the curves

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* Associate Mechanical Engineer, U. S. Office of Public Roads and Rural Engineering, Washington, D. C.

† "Elastic Hysteresis," by the author, *Engineering Record*, May 17, 1913.

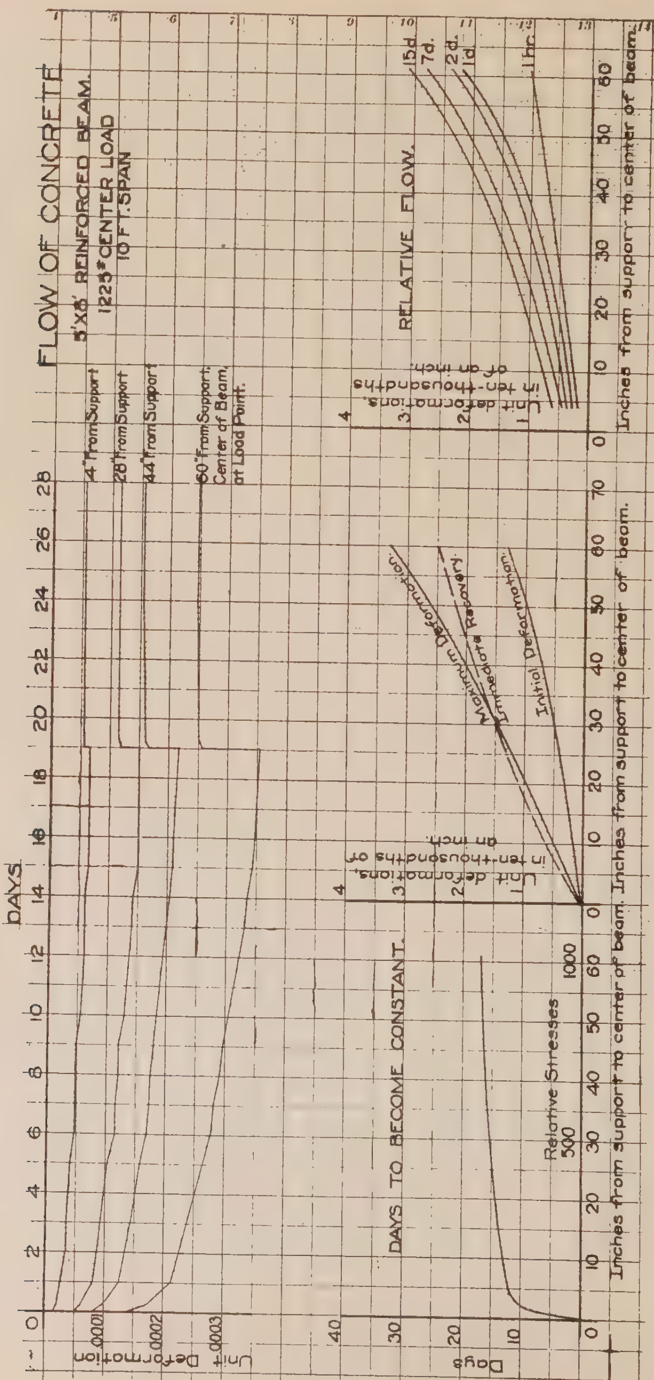

FIG. 1.—FLOW OF CONCRETE. 5 x 8-IN. REINFORCED CONCRETE BEAM; 1225 LB. CENTER LOAD; 10 FT. SPAN.

at the top of this figure. The load was sustained at the constant value of 1225 lb. for 19 days, and then released, deformations being read once each day. The first day the deformation readings were taken every two to four hours. These curves contain in their results the effect of the natural contraction of the concrete.

When the deformations become constant, the flow has attained its maximum value and has ceased. The time necessary for this to occur under the different stress values is shown by the curve in the lower left corner of Fig. 1. It should be noted that it has required about two weeks for concrete to attain its maximum deformation, with a fiber-stress of 500 lb., under the conditions of the test.

The group of curves shown in the lower center of Fig. 1, indicate the "initial deformation," the "maximum deformation," and the initial or "immediate recovery" of the beam, for the different stress values. The "relative flow" for the different stress conditions, at different elapsed times from the application of the load, is shown in the group of curves in the lower right-hand corner. These curves indicate the large increase of deformation which may be expected for short-time lapses.

The flow of concrete under a simple compressive load was obtained by testing a cylinder loaded to 700 lb. per sq. in. The cylinder was 6 in. in diameter and 24 in. long, of 1 : 2 : 4 gravel concrete. The strain-gage points were 20 in. apart, and readings were made once each day. A companion cylinder exactly like the first, and prepared in the same manner, was used, without being loaded, to obtain contraction data for correcting the deformations of the loaded cylinders, thus giving the net flow due to the load. These cylinders were cured for 29 days in wet sand, then allowed to dry out for 7 days before beginning the test. The results on these "dry" cylinders are shown by Curve A in Fig. 2, which gives the values of the "net flow." The curve is corrected for contraction effect. A second set of companion cylinders of the same size were tested in exactly the same manner, except that they were kept saturated. The flow results on these "wet" cylinders are shown by Curve B, Fig. 2, the "net flow" only being shown.

The deformation of concrete due to stress contains two factors, one which is probably somewhat elastic, and the other a molecular displacement of flow; the magnitude of the flow factor is dependent upon the time.

The determination of stress values in concrete structures from the deformation of the concrete is rather questionable, and entirely dependent upon the time factor. Stress values obtained from initial deformations, within only a few minutes after the application of the load, are probably more nearly true than later determinations. The value of the modulus of elasticity to be used in this connection should be determined on a specimen loaded at the same rate and with the deformations read after the same time lapse.

The effect of flow in most cases is to change or relieve the stress value; where this is impossible there is a gradual change in the length of the member, continuing possibly for several weeks.

In the case of a concrete beam with anchored or confined ends, the

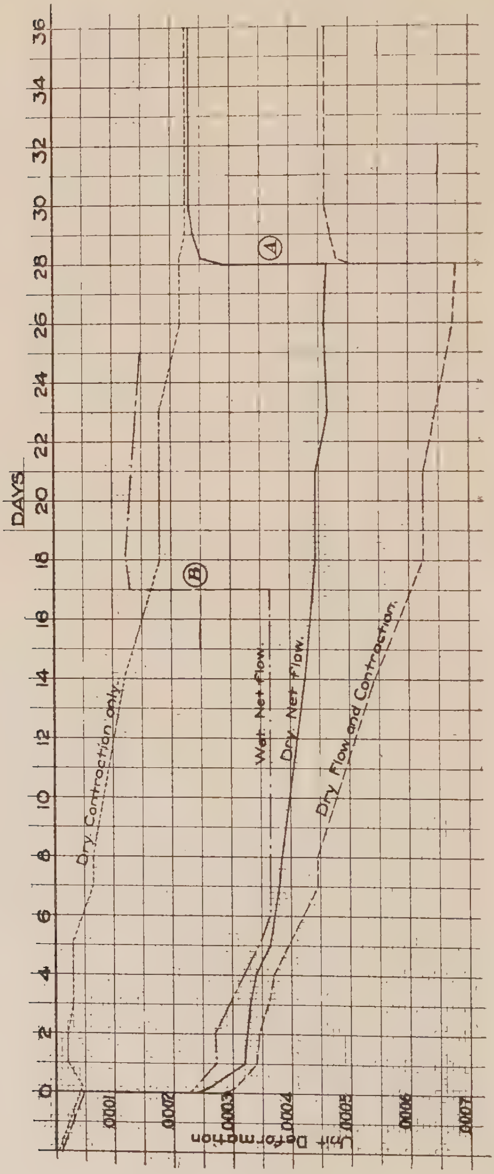

FIG. 2.—FLOW OF CONCRETE CYLINDERS 6 X 24 IN. LOADED 700 LB. PER SQ. IN.

temperature stresses are supposed to be very large; but since the temperature expansion is slow, there is time for flow to occur and thus relieve the stress. The amount of relief may be from 25 to 80 per cent of the computed temperature stress, depending upon the relative magnitude of the stress and the time factor.

This phenomenon of the flow of concrete may explain the abnormal stress conditions which apparently exist in some structures. Concrete roads and pavements are constantly subject to expansion and contraction, due to moisture and temperature, which in many cases do not crack where they apparently should. Concrete arches may also be largely protected from excessive temperature stresses by the flow of the material.

This subject needs much investigation to establish its exact effect and magnitude, before it should be considered as a factor in design. At present it should be used only as a possible explanation of certain deformation and stress phenomena.

DISCUSSION.

Mr. Wight.

Mr. Wight.—The tests by Professors Fuller and More and by Professor McMillan naturally recall to mind tests by other experimenters with other materials, the results of which were temporarily alarming to engineers. But it has been found in actual practice that the danger of permanent sagging and steadily increasing deflection of well-designed structures did not amount to much, and that the test results were to be considered more as warnings against flimsy construction than as evidences of defects in our methods of construction. We know of steel structures that are apparently so heavily loaded that many of their important members are stressed nearly to their probable elastic limit, and have been for years, and yet they show no evidences of insufficient strength. Such laboratory investigations as are given in the papers by these authors are valuable, but their results must be interpreted in the light of experience with actual structures, in order that we may see the experimental results in a true light.

Mr. McCullough.

Mr. McCullough.—In 1909 I investigated a number of warehouse floors build several years before from competitive designs in which the cost of the structures were kept as low as the designers felt it was safe to go. In some of these buildings we found a continual increase in the deflection of floors under constant loading, although there were no failures. On one occasion I had an opportunity to make a test lasting eleven months which showed very clearly that continually increasing deflection could be produced without early failure. A slab was supported so that its two ends cantilevered out from the supports and was loaded until the stress in the steel reached the elastic limit. During the last three months the increase in deflection was rapid, indicating that the steel was showing the effect of the continued strain and the concrete may have been compressed at the same time. When the test was stopped because the room was needed for other purposes, the concrete over the points of reverse moments showed cracks along the top of the slab, and the slab recovered only about 45 per cent of the deflection after the load was removed.

I have had an opportunity of making a number of tests of small beams and thin slabs, using the exact details employed by the persons who prepared competitive designs and applying test loads approximately those for which the designs were made. These loads were left on the test pieces from 3 to 16 months and showed a decided increase in deflection as the tests continued.

Professor Hatt.

Professor Hatt.—In investigations of plastic flow considerable light can be obtained from the behavior of wood under continued loading. With that material the experience of many years indicates that if the stress does not exceed the elastic limit the plastic flow finally comes to a halt, but if the load exceeds the elastic limit the material eventually fails.

PROFESSOR MCMILLAN*.—I cannot agree with the criticism just made that the conditions described in the paper are not observed in practice. The conditions exist but are not observed any more than the cracks in stucco are observed. Owners of buildings say that there are no deflections in them, yet deflections will be found in many of them if measurements are made to find out if they exist. In examining numerous buildings of different types, evidences of deflection were found in a very large proportion of cases. Sometimes poor design, inferior materials and bad workmanship may have been contributing causes, but surely not in all. **Prof. McMillan.**

MR. SMITH.—Perhaps the best answer that can be made to Mr. Wight's statement regarding the application of the results of laboratory tests of plastic flow to actual conditions is to refer to an incident that occurred in the course of litigation. Several specialists were called upon at different times to measure the deflections in a building. Each succeeding report showed larger deflections than those previously stated, which was undoubtedly due to this phenomenon. It should be pointed out, however, that plastic flow does not necessarily tend to induce failure, for it is perhaps more likely to relieve stress. Furthermore, plastic flow, using the term in a sicentific sense, does not exist in ferrous metals stressed below their elastic limit. It occurs in the concrete and it must cause a change in the distribution of stresses in a reinforced-concrete member where it takes place. It is the absence of any consideration of plastic flow which has clouded the studies of tests of flat slabs; in fact, by loading such a floor and then waiting long enough for plastic flow to produce the desired results, it is practicable to prove a good many theories of flat slab behavior which do not take into account the phenomenon under consideration. **Mr. Smith.**

* Professor McMillan discussed orally at the same time the papers by Professors Fuller and More and by Mr. Smith, but he submitted for publication the written discussion of the former paper, which is printed on page 311.

TESTS OF LARGE REINFORCED CONCRETE SLABS.

By A. T. Goldbeck* and E. B. Smith.†

In June, 1912, an investigation was started at the U. S. Office of Public Roads to determine the distribution of stress in reinforced concrete slabs subjected to concentrated loads, and a report of the first six slabs tested was made in June, 1913, before the American Society for Testing Materials.‡ These preliminary specimens were 9 ft. wide with a span of 6 ft., excepting one which was 6 ft. wide with a span of 12 ft.; the effective thickness varied from 3 to 6 in. Although these tests were not conclusive, they gave valuable results which perhaps had some influence in governing the subsequent design of bridge slabs. In addition, they indicated the desirability of making more complete tests, and suggested to others the importance of continuing investigations along similar lines.

During the past two years there have been tested three large-size reinforced bridge slabs built on specially constructed equipment, and it is the purpose of this paper to briefly describe these tests.

The object in view is to secure data concerning the distribution of stresses in flat reinforced concrete slabs to serve as a basis for reasonable assumptions in the design formulas. Whenever the design of a slab is undertaken, some assumption for the width value (b) must be made for substitution in the design formulas used in computing the thickness and the reinforcing. The proper selection of this value has been subject to a great deal of uncertainty, since a wide range of widths have been used in slab designs. The correct design of a slab involves the selection of such a width that its resisting moment (assuming that the stress is uniformly distributed) will be the same as the resisting moment of the whole slab under actual stress conditions. To simplify this statement, it needs only to be remembered that the resisting moment is a direct function of the fiber stresses or of the fiber deformations. Therefore, the width to be selected should be such that the total maximum fiber stress uniformly distributed will be the same as that of the whole slab whose maximum fiber stress is *not* uniformly distributed from the center of the slab to the two free edges. This width when so selected is known as the *effective width*; and its value should be based upon experimental data, from which it may be obtained by dividing the maximum fiber stress into the summation of the maximum fiber stresses along the center line parallel to the supports.

A value for the effective width may be obtained from the fiber deformations of the concrete, and another value from the deformations of the reinforcing steel. A few examples of the difference in these values are shown in the accompaying curves of Figs. 4 and 5. It may be seen that the effective

* Engineer of Tests.

† Associate Mechanical Engineer, U. S. Office of Public Roads and Rural Engineering.

‡ "Tests of Reinforced Concrete Slabs Under Concentrated Loading," by A. T. Goldbeck, 1913 *Proceedings*, American Society for Testing Materials.

width values obtained from the concrete deformations are the smaller, and since there is a difference in the values obtained by these methods, it is obviously proper that the method giving the most conservative value for the effective width is the one which should be used. Thus, if the stresses in the steel showed a more critical change than in the concrete at the center of the slab, or at the load point, then we should be justified in basing our evaluation on the steel stresses or deformations. This, however, does not seem to be the case, as shown in Figs. 4 and 5, where it may be seen that the local effect of the load is more pronounced in the concrete than in the steel.

Each of the three slabs herein described was made of 1:2:4 gravel concrete, machine mixed, and was reinforced with 0.75 per cent of plain square steel bars with no transverse reinforcing. The slabs were 32 ft. wide, of 16 ft. clear span, with a 10-in. bearing width on each support.

The forms were removed from the slabs at the end of two weeks, and each slab was cured in air under approximately the same conditions, within a building constructed especially for housing these tests; the slabs were sprinkled once each day until the forms were removed. Tests were begun at the age of about 28 days and were continued periodically until the time of

TABLE I.—DIMENSIONAL DATA.

Serial No.	Thickness.		Reinforcing.			Modulus of Elasticity, lb.	Breaking Load, lb.
	Total, in.	Effective, in.	Size, in.	Spacing, in.	Per Cent.		
835	12	$10\frac{1}{2}$	$\frac{3}{4}$	10.5	0.75	2,900,000	119,000
930	10	$8\frac{1}{2}$	$\frac{3}{4}$	8.87	0.75	4,000,000	80,000
934	7	6	$\frac{1}{2}$	5.56	0.75	3,000,000	40,000

breaking. Slab 835 was broken at the age of about 6 months; slab 930 at about 5 months; and slab 934 at 3 months. The breaking loads are given in Table I.

The deformation values for each slab and for the different load values are shown graphically in Figs 1 to 5. Each curve is plotted from the deformation values measured perpendicular to the supports and spaced along the center line parallel to the supports. It will be noted that the general shape of the curves for all loads is the same for the concrete and the steel. The local effect of the load is also shown very clearly.

A series of deflection curves for the last slab tested (No. 934, 6-in. effective thickness) is shown in Figs. 6 and 7. The maximum deflection for the working load was 0.3 in. at the center.

One of the important facts which these tests have brought out and which this paper seeks to impress upon concrete engineers, is that the time element is a very large and important factor in determing stress values and their distribution from the fiber deformations. Nearly all materials of construction with which the engineer has to deal will yield an approximately constant relation, independent of the time, between the load and its accompanying

deformation within the limit of elasticity. This is not true in the case of concrete, which exhibits a marked flow or molecular adjustment under working stresses extending over time periods of several weeks. During the tests of one of these slabs in August, 1914, this phenomenon was first discovered, and a series of experiments was begun to obtain data regarding

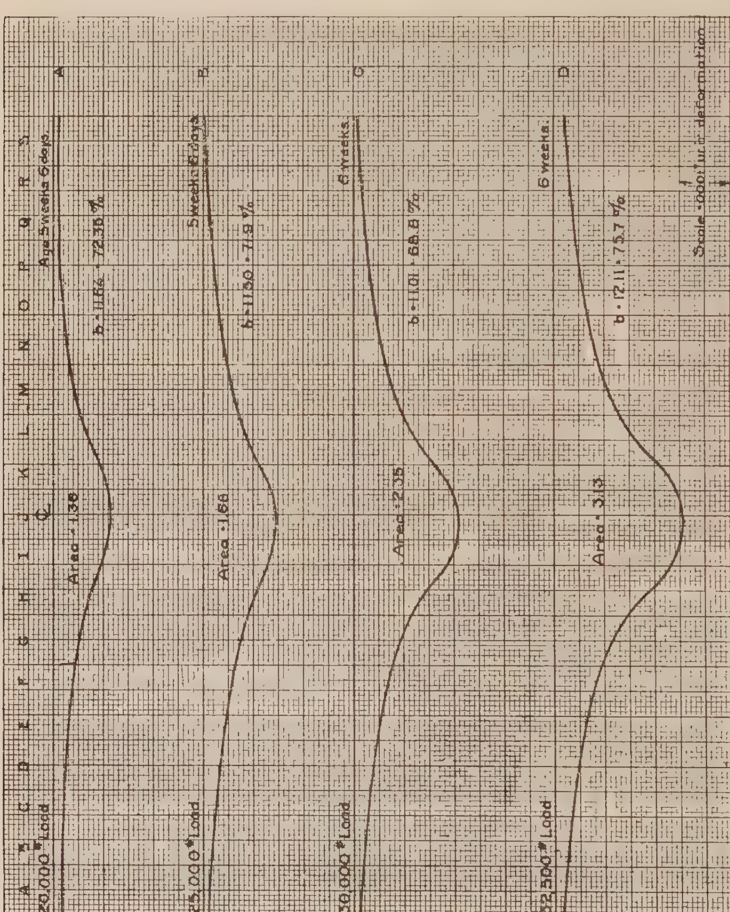

FIG 1.—SLAB NO. 835. CONCRETE DEFORMATION CURVES. CONCENTRATED CENTER LOAD. EFFECTIVE THICKNESS, 10.5 IN. ALL FROM SAME ZERO.

the flow of concrete. Since it is not the object of this paper to discuss the facts or the theory of this phonemenon, the following statement will be sufficient for the present purpose.

When concrete is subjected to a compressive fiber stress of 700 lb., the immediate fiber deformation is only about 50 per cent of what it will be if

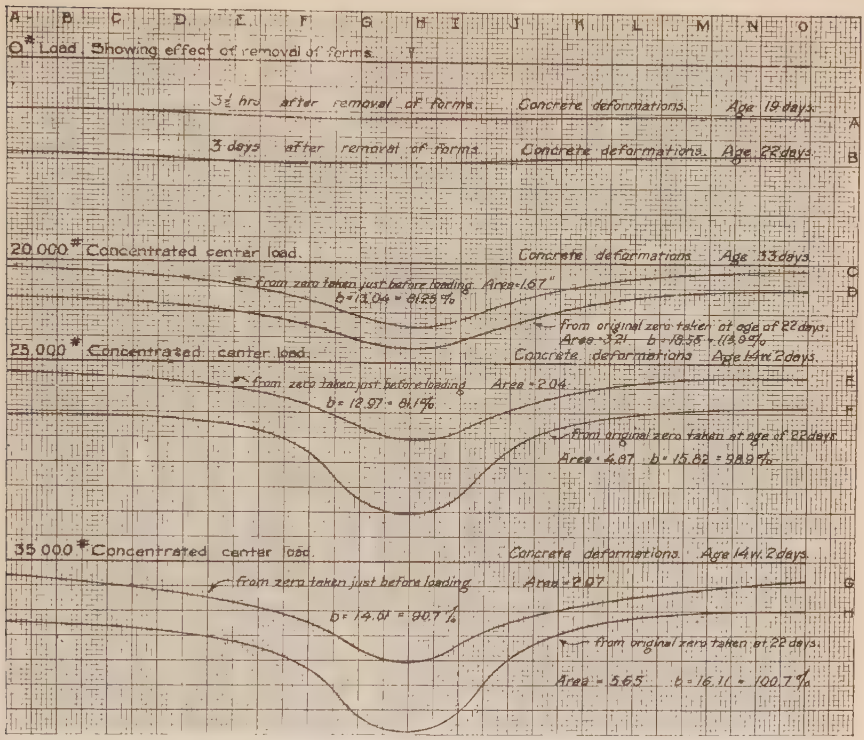

FIG. 2.—SLAB NO. 930. CONCRETE DEFORMATION CURVES. EFFECTIVE THICKNESS OF SLAB $8\frac{1}{2}$ IN.

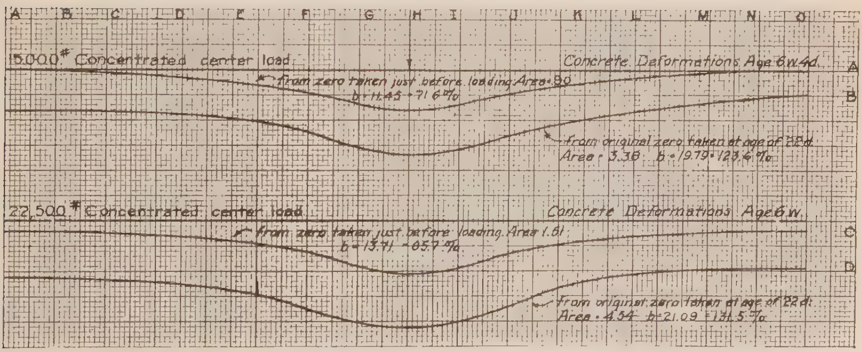

FIG. 3.—SLAB NO. 930. CONCRETE DEFORMATION CURVES. EFFECTIVE THICKNESS OF SLAB $8\frac{1}{2}$ IN.

the stress is maintained for three weeks. Within 24 hours after the application of the load the deformation has increased an additional 20 per cent; during the first hour the deformation may show a change of 5 per cent or more. Furthermore, the recovery of the deformations, after the removal of the load, is slow and not complete.

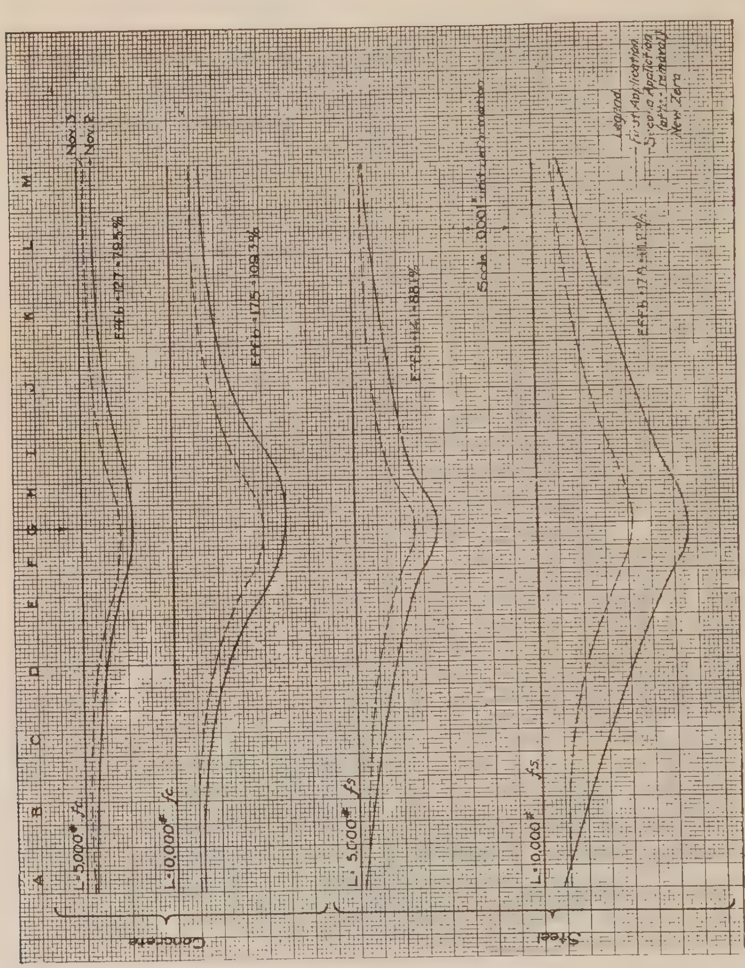

FIG. 4.—SLAB NO. 934.

This fact is mentioned here to show the importance of taking all the deformation readings at approximately the same time after the load is applied, and in each case a new set of deformation readings under zero load must be taken just before each load application.

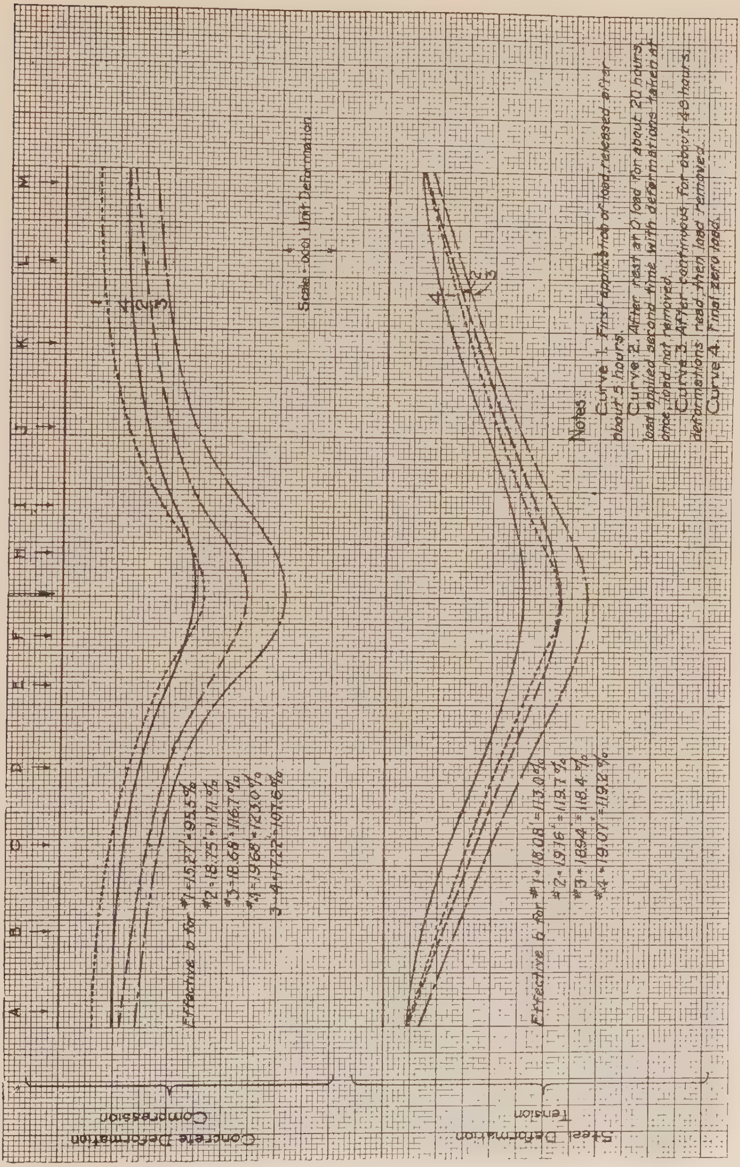

FIG. 5.—SLAB NO. 934. DEFORMATION CURVES. COMPUTED FROM THE FIRST ZERO READING. 10,000 LB. CONCENTRATED CENTER LOAD.

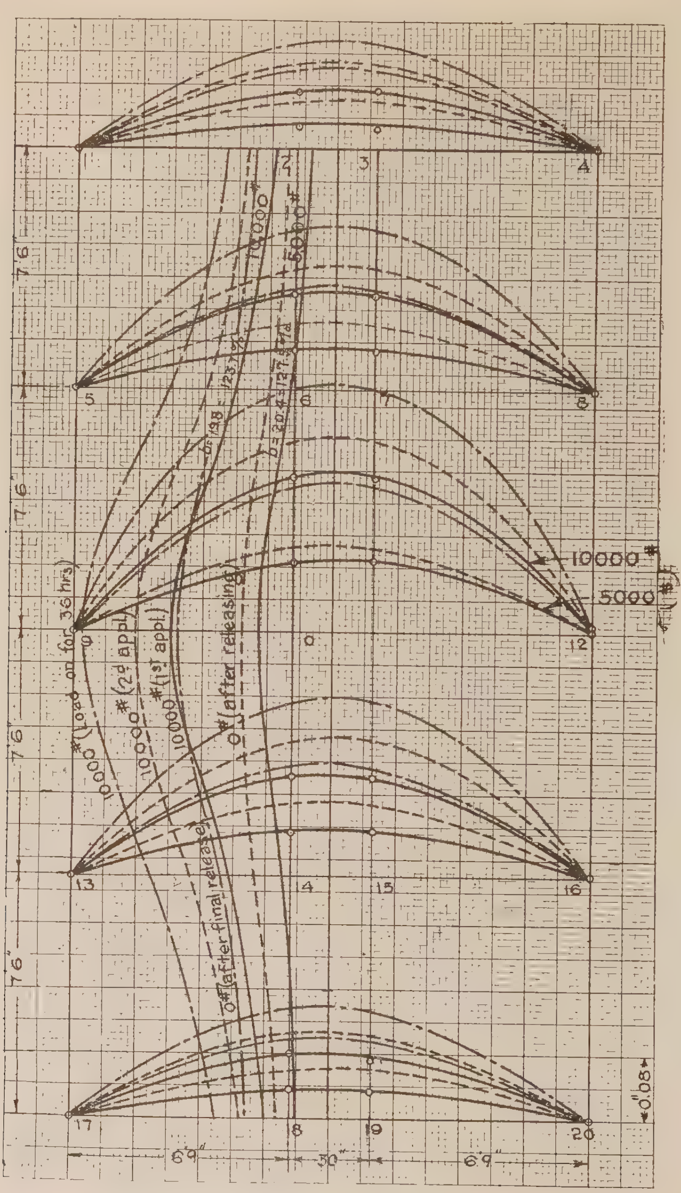

FIG. 6.—SLAB NO. 934. DEFLECTION CURVES.
TEST OF NOVEMBER 2, 1915. (FIRST APPLICATION OF LOAD.)

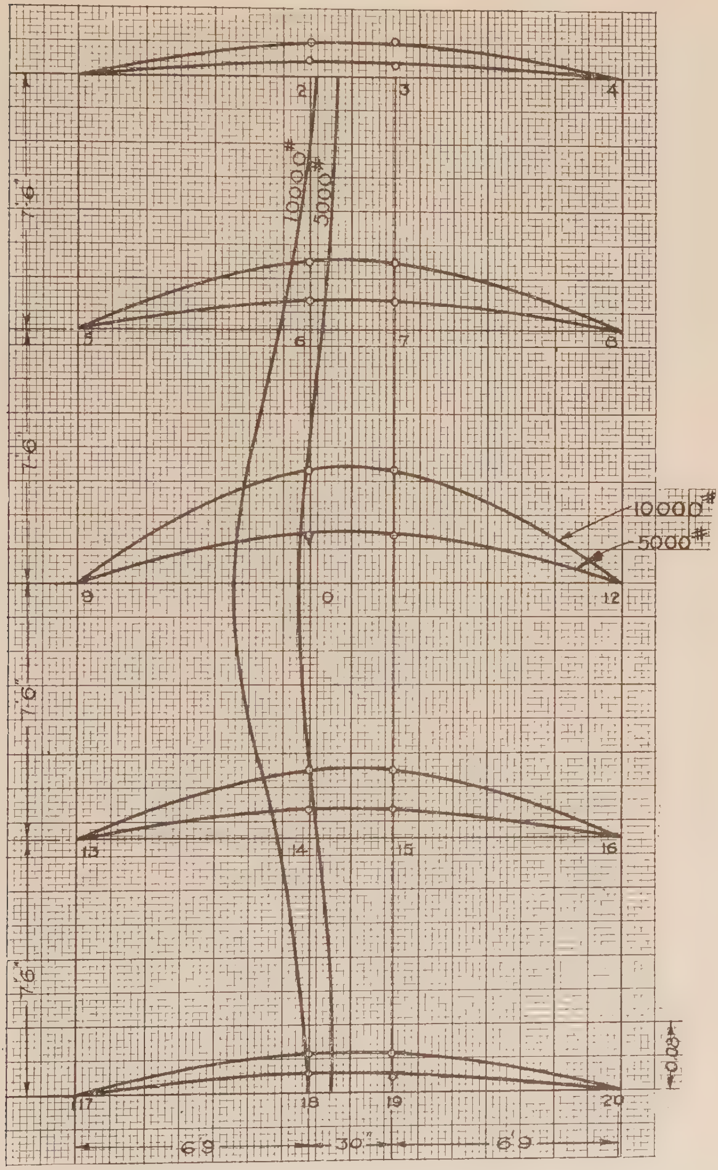

FIG. 7.—SLAB NO. 934. DEFLECTION CURVES.
TEST OF NOVEMBER 3, 1915. (SECOND APPLICATION OF LOAD.)

The effect of time on the value of the deformations is shown in Figs. 2 and 5. For instance, it may be noted by reference to Fig. 2, curves E and F, that there is a deformation change of about 100 per cent at the center of the slab and nearly 400 per cent at the edges, between the values obtained from a new set of zero readings, taken just before the application of the load, and those obtained from an earlier zero reading taken a few weeks before, with the application and removal of several loads intervening. Also it should be noted that the value of the effective width is increased nearly 22 per cent. The direct effect of this time factor is shown in Fig. 5, curves 2 and 3 (concrete deformations); here the load was sustained for 48 hours, and the center deformation increased about 20 per cent.

Many other interesting time and set effects may be observed by a detailed study of these curves, but the authors' purpose is here accomplished by emphasizing that such tests, and in fact all tests for stress and deformation

TABLE II.—EFFECTIVE WIDTHS.

Slabs, 16 ft. span by 32 ft. wide.

Center Load, lb.	Slab 835, 10½ in. Effective Thickness.	Slab 930, 8½ in. Effective Thickness.	Slab 934, 6 in. Effective Thickness.
15,000	....	11.4 ft.: 71.6 per cent of span	12.7 ft.: 79.5 per cent of span
20,000	11.6 ft.: 72.3 per cent of span	13.0 ft.: 81.2 " " " "	17.5 ft.: 109.3 " " " "
25,000	11.5 ft.: 71.9 " " " "	12.9 ft.: 81.1 " " " "	....
32,500	12.1 ft.: 75.7 " " " "	....	....
35,000	....	14.5 ft.: 90.7 " " " "	....
Safe load	12.1 ft.: 75.7 " " " "	12.9 ft.: 81.1 " " " "	17.5 ft.: 109.3 " " " "

values in concrete, must be conducted only with a full realization of the importance of the time factor. The *immediate* fiber deformatiom is the value which should be used, and not the value obtained after a long-time suspension of the load, nor that which contains the effect of several applications and removals of loads.

The determination of the effective width value for use in the design of flat slabs supported at two edges, is attended by so many variables that, from the information at hand, a final and definite formula for its determination cannot be given. Its value, however, is affected by the span, the total width, the thickness, the load on the slab, the distribution of the loads, and probably by special forms of reinforcing. The thickness and the load with its distribution are the only variables that have been considered in the tests thus far, and the values for the effective widths are collected in Table II.

From these values it may be noted that there is a general tendency for the effective width to increase slightly with the increase of load. Also the effective width seems to vary inversely as the thickness of the slab.

In the light of the information available at the present time, we should be safe in using a value for the effective width equal to 0.7 of the span. This will probably result in the design of a somewhat thinner slab than is usual;

but the fiber stress values and the large ultimate breaking loads of these slabs are an indication of the safety of such designs.

This series of experiments has not been completed, but is being continued in the hope of arriving at a definite formula for determining the value of the effective width as influenced by all of the important variables.

DISCUSSION.

Mr. Westergaard.

Mr. Westergaard.—The results of the tests described in this paper seem to indicate that a distinction must be made between tests in which a constant load is applied, with a resultant continuous increase in deflection and tests in which, after a certain deflection is reached, the load is gradually decreased in order to maintain the deflection at the amount. In the former case the time-effect will be an increase in deflection and in the latter case a decrease in the load. As a rule, it is more easy to weigh the load immediately after its application than to measure the deflection at that time. So in order to obtain comparable results, it is apparently preferable to produce a deflection rapidly and then maintain it, rather than to maintain a constant load and measure the deflections.

Prof. McMillan.

Professor McMillan.—There is this practical objection to Mr. Westergaard's proposed method of testing, that in some tests not only does the load decrease over night but the deflection increases, and the investigator must employ results which give the balance between these two factors. Sometimes the load will decrease without any increase in the deflection. The results seem to depend on the relation of the applied load to the ultimate strength of the concrete in compression.

Mr. Smith.

Mr. Smith.—Mr. Westergaard apparently overlooks the fact that concrete may show a change in deformation without any change in stress. In a test made in a machine where the load is applied by a screw, the machine does not continue to apply the load as deformation continues, but if a dead load is used in the test, it remains constant while the deformation increases. The deformation resulting in a test made with a screw machine will be different, therefore, from that produced in a test with a dead load. The stresses computed from these tests would also be very different, because there may be a deformation in the concrete which will not indicate any stress at all.

REPORT OF COMMITTEE ON FIREPROOFING.

The most important item of the work before this committee is the question of fire protection for columns in fire-resisting buildings. This necessarily follows from the structural importance of the cloumn, and the disastrous results if it fails.

There is about to be undertaken at the Underwriters' Laboratories in Chicago, a very elaborate and carefully planned series of tests on column coverings. It seems probable that this series of tests will yield a greater amount of authoritative information than has been accumulated from all tests and fires up to the present time.

It has seemed to the committee that it would not be wise at this time to attempt to draft standard specifications with this series of tests pending.

Probably every member of the committee, and many members of the Institute, believe that specifications could now be drawn which would be a great improvement over current practice. But such opinions are necessarily based upon the data hitherto available; no one has been a witness of all the tests, nor has anyone studied, on the ground, the results of all important fires in actual buildings. Existing opinions are therefore based upon data not only incomplete, but generally resulting from conditions not admitting of rigid comparison.

Owing to the paramount importance of the column, and to the imminence of these important tests, the committee think it best to report progress and suggest that their work be continued.

There has been considerable correspondence and a number of personal interviews between members of the committee, but no meeting of the committee as a whole, because the situation did not seem to warrant it.

Views have been interchanged, fundamental principles discussed, and collateral data are being collected. But it seems premature to go into details at this time.

It is hoped that the committee will be able at the proper time not only to submit standard specifications worthy of adoption, but that these specifications may set a higher standard, and that the collateral data may enable the committee to prove that the adoption of a higher standard into current use is thoroughly justified as a commercial proposition. It is felt that not only should the standards of good practice be defined, but the way should be made clear for their acceptance by the owners of commercial buildings, for sound business reasons.

As the committee is dependent upon the courtesy of other persons for much of the data needed to demonstrate commercial practicability, this part of the work can not well be hurried.

For the reasons set forth, the committee feel that they must be content, at this time, with this brief statement of the end towards which they are working, and of the fact that the work is actually in progress.

Respectfully submitted,

JOHN STEPHEN SEWELL, *Chairman*,
EDWIN CLARK,
IRA H. WOOLSON,
CHARLES L. NORTON,
W. C. ROBINSON.

February 2, 1916.

Mr. W. C. Robinson,
Underwriter's Laboratories,
Chicago, Ill.

DEAR SIR:

I have gone over the schedule of proposed column tests with some care. Professor Woolson kindly furnished me with a copy of his comments addressed to Mr. W. W. Stratton, which you no doubt have seen.

It appears to me that the series of tests is worked out with extreme care, and I hesitate to comment on it, especially in view of the fact that my own experience in devising such tests is very limited. Of course, I hold opinions, but they are based on observation of actual occurrences where the factors determining the results were variable and more or less unknown, so that the results are not comparable, in the strict sense of the word.

With this explanation, the following comments are submitted:

(1) If this series of tests is to be made exhaustive, the schedule of unprotected columns seems a desirable feature. It should yield the fundamental data from which to build. If the tests are not to be numerous enough to be exhaustive, possibly more valuable results might be secured by using some of the proposed unprotected columns for duplicate protection tests.

I thoroughly approve of testing a variety of concrete aggregates; I cannot help believing that a concrete using broken bricks or terra cotta, or good hard clinkers, will prove much more fire resisting than one using gravel or broken stone; and am inclined to think that gravel may show a measurable superiority over many kinds of broken stone—but on this point I have mental reservations.

(2) My present business has made it necessary for me to study very carefully the subject of expansion cracks in stone deposits—including the effects of contraction and the strains set up by both expansion and contraction. I am led to believe that an expansion crack once started, is due to a strain which may remain and slowly extend the crack, even after the exciting cause is removed. This may be of importance in reinforced-concrete

members, and might be the determining factor in deciding whether, after a fire, repairs should be attempted, or whether total renewal was necessary.

(3) It appears to me that, in case the tests should show any of the common methods of column protection to be inadequate, we should be prepared to point the way to adequate methods without a prohibitive increase in cost. It has often occurred to me that the stiffening effect of a solid concrete filling, amounting to a virtual increase in the radius of gyration, might justify such a saving in steel as to offset an increase in the cost of the covering. The tests on partially protected columns ought to throw light on this, provided they can be made also to yield reliable data as to the stiffening effect.

(4) Commercial hollow tiles, in the sizes commonly used, when laid in mortar and rigidly restrained, can set up expansion stresses in the individual tiles sufficiently severe to spall off the exposed webs and often destroy the covering. If possible, I would like to see four columns added to the series, protected with common bricks laid in the bed and also on edge, and with ordinary low grade fire bricks used in the same way. Some observations I have made in actual instances, lead me to believe that such protection would be better for the column, and in less need of repairs itself after a fire, than most of those in common use.

(5) I believe that if we could devise a form of covering that will not participate in the structural duties of the column, it will be more efficient, because the temperature stresses will not be added to others already in existence. In this case, cracks that might destroy a reinforced concrete column for example, as a structural member, might not destroy the efficiency of the covering for fire protection. Whether your tests can be made to throw light on this I am unable to say, but I believe it is an important point. The idea suggests itself that the fire protective covering might be divided into two parts, separated by a plane of weakness; the other covering to be sufficient to resist moderate fires; the plane of weakness to stop expansion cracks; and the inner covering (or filling) together with the damaged outer covering to be depended upon for protection in the latter stages of a severe fire, assuming that the whole covering would have to be removed only after such severe tests.

(6) When we have had a fire, it is important to determine proper methods for restoring the structure fully to its original degree of efficiency from both a structural and a fire resisting point of view, at a minimum cost; in other words, to get a measure of the actual damage in dollars and cents, including the value of the time consumed in repairs—or, if you like, a measure of the salvage. This question bears very directly upon the commercial practicability of better methods. It has not been much studied by the underwriters, so far as I know, hence I infer that the rates in existing fireproof buildings have yielded a good profit, or else that experience with such buildings has not been studied in sufficient detail to show whether they yield the underwriters a profit or a loss. Of course, in such a study, the conflagration experience should be included, and so should the question of the contents of such buildings, and the methods actually adopted in restoring

them. The owners should be penalized in the rates for inadequate repairs. Has that been done? Of course, any study of this subject leads to the conclusion that fire prevention is much more important than fire protection; but the latter cannot be ignored, and if possible, the present series of tests should go beyond the question of the structural survival of the column, and include some consideration of the repair work which follows a test by fire.

(7) My reasons for thinking that gravel may be better than broken stone are that gravel consists of particles which by long exposure have yielded to temperature stresses until planes of weakness and lack of homogeneity have been largely eliminated. The sudden stresses due to crushing do not permit of such results in broken stone, and I should anticipate a good deal of spalling in the pieces of stone themselves, when the fire test is applied. But gravel concrete may develop other weaknesses, so I do not feel sure of the result. In this connection I have been much impressed with the chapters in standard works on geology which treat of the weathering of stone in hot climates. A study of such matters is very suggestive to one interested in fireproofing. I am writing this from Pittsburgh. As soon as I get home I will look up and send you some references. In the meantime, if you could interest Prof. Salisbury, of the University of Chicago, in your tests, I think it would be worth while. He is an eminent geologist of long experience in both tropical and temperate climates, and can tell you a great deal about the spalling of stone deposits when exposed to a hot sun. In my judgment, taking the world as a whole, stresses due to high temperatures are more potent in disintegrating rocks than the frosts of higher latitudes.

(8) This letter is getting too long, but I think a study of kilns and furnaces is not without its value to fireproof engineers; and an authoritative grading of different forms of fireproofing as to efficiency will be a potent means of forcing better methods into commercial use, even at an increased cost.

I realize that much of what I have said is quite general and not very specific, but I hesitate to suggest specific changes in your admirable program, and believe you are better qualified than I am to determine the specific bearing of my somewhat general discussion in the program itself.

Very truly yours,

JOHN STEPHEN SEWELL.

PROGRESS REPORT, COMMITTEE ON INSURANCE.

The opinion of the Insurance Committee of the American Concrete Institute is that the most effective work it can do is to collate authoritative data regarding the behavior of reinforced concrete buildings, particularly industrial buildings, under actual fire conditions. It does not feel that any work within its limits on insurance rates or in tabulating comparative rates will be of much value, as the subject of fire underwriting is exceedingly complicated and requires long experience and much time properly to interpret.

The committee has taken no active steps toward collecting information on the fire record of concrete buildings, chiefly for the reason that it recognizes the large amount of clerical and detail work involved by a proper investigation of this kind, and it has no clerical force at its disposal to carry on such an undertaking. It has seemed best to wait until the Institute's executive office was so organized that the stenographic and clerical burden could be assumed by it.

The program which the committee would like to see carried out is as follows:

1. Correspondence with the proper underwriting associations or insurance bodies with a view of getting from their files complete records of concrete buildings in which there have been fires.

2. In the event of discovering that the insurance companies or associations' records were not kept in such manner as to secure this information readily, or in the further event of the committee not meeting with a spirit of co-operation on the part of the fire underwriters, the plan would be to endeavor to persuade all the cement manufacturers to search their files and report to the committee, as it would seem they should be able to do, a list of all buildings which were built with their cement in which fires had occurred.

3. In requesting this record of fires, the chairman's judgment is to limit the investigation to fires where damage of $20,000 or more had resulted.

4. Upon receipt under either of the above described schemes of the record of fires, this record would:

A. Be classified by states.

B. Be classified by character of buildings reported (that is, industrial, loft, hotels, residences, etc.).

C. Be classified by fire damage:

$20,000 to $50,000
$50,000 to $100,000
$100,000 to $200,000
Over $200,000.

5. After classifying the record of fires, the industrial buildings with the most serious losses by fires would be picked out and a form letter drafted to

be sent to the owners of the respective buildings. This letter would inquire among other things as to:

A. The origin of the fire;
B. Nature of fire;
C. Duration of fire;
D. Damage to the contents;
E. Damage to the building in detail;
F. Was building sprinkled?
G. General character of fire protection and fire prevention available;
H. Cost to repair damage to the building;
I. Request for photographs, if available;
J. Insurance rates applying on building (any increase in rates as result of fire);
K. Comments by the owner on how concrete stood up;
L. Would the owner adopt concrete for a similar building after his fire experience;
M. Name of insurance company (the idea being proposed at some future date to secure an opinion of concrete's fire resistance value from said insurance company).

6. If this inquiry on the most interesting class of damaged buildings meets with favorable and worth-while replies, a similar letter would be sent to the other classifications.

7. If the returns from this investigation were unsatisfactory, the committee will determine whether to follow up the original letter with a further appeal or whether to abandon the investigation as impracticable.

8. Assuming satisfactory response to the form letter, there would arise the need of classification of the replies and detailed study of the resulting record, the idea being to show, if such exists, some relation in percentages or by diagram, between the total fire loss in a given fire, the damage to contents and the damage to the structure, and the relation of these factors to the original cost of the building and to the cost of insurance.

9. Upon digest of the reports and the forming of an opinion by the committee, it might be advisable to endeavor to persuade the chief underwriting associations to appoint a committee to meet with this committee and formulate, as a result of such meeting, a report which would be of interest to the fire underwriters, it being hoped that the nature of such a report would be such as to warrant underwriting bodies to take steps to so alter existing rates as to favor reinforced concrete for building construction more is now the case. Regardless of co-operations of this kind with the underwriting bodies being brought about, the committee would submit to the Institute its report and recommendations. Assuming the report of value, it would seem advisable to distribute it to a large and live list of underwriting officials.

As to the expense of carrying out this, there are something like 150 underwriting associations or exchanges keeping records such as the committee desires access to. Also there are about 100 cement manufacturers to whom it could apply.

The stenographic work in writing the original letter to these associations or concerns should not cost over	$125.00
Postage would amount to	25.00
Total initial expense	$150.00
Clerical work classifying list of fires	100.00
Stenographic work preparing form letter to the owners of buildings where fires occurred	$50.00
Postage	10.00
Classification of replies received from the fire owners	25.00
Miscellaneous correspondence	25.00
Stationery	40.00
Committee expenses for probably three meetings, telephone charges, etc.	200.00
Contingency allowance	300.00
Total appropriation which in the committee's judgment is necessary to carry through successfully this program (assuming information is of practicable attainment)	900.00

It will be apparent from a study of the above that the preliminary expense necessary to make a start on such an investigation is small, as it consists of paying for the initial correspondence to see if we can obtain a satisfactory record of concrete building fires. It is not, in the chairman's judgment, at least, a matter so much of expense as it is of proper office facilities for correspondence, filing and classification. The members of this committee are all busy men and cannot draw on their own office staffs for this work, and the chairman does not feel like urging them to do so.

Since the above was written, Mr. F. W. Moses has suggested that the committee make another classification for those fires occurring in which the damage is less than $20,000. Mr. Moses is of the opinion that if this classification is not made that a large part of the benefit derived from concrete construction and watertight floors will not be revealed.

J. P. H. PERRY, *Chairman.*

DISCUSSION.

Mr. Turner.

Mr. Turner.—It may prove easier to obtain a list of all the concrete buildings in the country by applying to the contractors for such structures than by trying to secure the information through cement companies. I cannot agree that it is desirable to limit the cases of fire damage to be considered to those where the loss is at least $20,000. We do not expect material losses to follow fires in concrete buildings, and if nothing is to be said about cases where the loss does not reach $20,000 we will be throwing out of consideration some of the best evidence we have of the value of concrete construction. There have been many fires in such structures where the total loss has not exceeded $200 or $300, but which would have been much greater had a different type of construction been employed.

Mr. Wig.

Mr. Wig.—It should be stated that a rather complete collation was made several years ago of all the literature on fires in concrete buildings. A voluminous report of about 500 or 600 pages was prepared for the Committee on Concrete and Reinforced Concrete of the American Society of Civil Engineers, which can probably be obtained by your committee from Mr. Richard L. Humphrey, who was secretary of this committee, or from the Portland Cement Association, which I believe has a copy in its files.

REPORT OF THE COMMITTEE ON NOMENCLATURE.

At the last Annual Convention of the Institute, the Committee on Nomenclature submitted a report which included a preliminary list of definitions of the more common terms used in concrete work. There were a number of criticisms of these definitions on the floor of the convention, and in accordance with such criticisms and some later correspondence, the committee revised the list of definitions to the form which was presented in the April, 1915, *Journal* of the Institute. Since the publication of that number, there has been received but one communication from a member of the Institute. After consideration of this communication, the committee begs leave to submit the following revision of the previous list of definitions:

Omit the definition "Rerolled steel—Steel bars rerolled from old material," and substitute in its place the following two definitions: ("Billet Steel Bars—Bars rolled from new billets;" "Rail Steel Bars—Bars rolled from standard section T-rails.")

With this correction, the committee submits the definitions to the Institute for final acceptance.

The committee has seriously considered during the past year an extension of the list of definitions, but it feels that for two reasons the time is not right for such extension.

(1) A list of definitions prepared by a small group of men, such as a committee, obviously cannot represent a general consensus of opinion. The only proper method for the preparation of such a group of definitions is for a committee to prepare a preliminary list which should be submitted to the closest scrutiny of a large number of men expert in the work to which the definitions relate. Inasmuch as the members of the American Concrete Institute have not felt it at all necessary to criticize either adversely or favorably the definitions submitted by the committee, and further, inasmuch as the committee realizes that such definitions cannot have the perfection that such lack of criticism would imply, the committee feels that there is not sufficient interest in the preparation of definitions to continue the rather arduous labor of preparing them.

(2) Fundamentally, it is wrong that one society, however expert it is, should attempt to promulgate a set of definitions on subjects in which many other societies are interested. The proper method for the preparation of such definitions is through a joint committee. The Committee on Nomenclature of the American Concrete Institute therefore recommends to the Institute that it make every effort through its executive office to interest other societies, such as the American Society of Civil Engineers and the American Society for Testing Materials, in the appointment of a Joint Committee on Nomenclature, to which the American Concrete Institute may send delegates. Only by the joint acceptance of such definitions can any real progress be made.

F. C. Wight, *Chairman*,
Peter Gillespie, E. J. Mehren,
F. E. Turneaure, L. R. Ferguson.

DISCUSSION.*

Mr. Chamberlain.

Mr. Chamberlain.—Before the Institute votes to adopt this nomenclature, it will be well for us to help the committee by checking the proposed definitions with those given in treatises on concrete. It is undesirable to confuse the student and the layman with new definitions of old terms unless there are good reasons for the new definitions. This is merely another way of stating the thought expressed in the last part of the committee's report.

Mr. Chapman.

Mr. Chapman.—We surely should not vote on such a very important matter until an opportunity has been given us to refresh our memory by reading the definitions. They are one of the most important subjects that technical societies deal with, and should be compared with the definitions already made in this and other fields employing similar materials or using some of our terms to designate other materials. It should not be overlooked that definitions adopted by the American Concrete Institute may be used by lawyers and interpreted by courts in a way no engineer would think of doing, unless their meaning is perfectly clear to an untechnical man.

(The President requested Mr. Ferguson to read the proposed definitions, which are:)

I. Concrete.

An artificial stone formed by the mixture of hydraulic cement with water and an aggregate composed of hard inert particles of various sizes.

Reinforced Concrete.—Concrete in which metal (generally steel) has been imbedded in proportionately small sections in such a manner that the metal and the concrete assist each other in taking stress.

Rubble or Cyclopean Concrete.—Concrete in which large stones are imbedded after mixing and during placing.

II. Cement.

Portland Cement.—The finely pulverized product resulting from a calcination to incipient fusion of an intimate mixture of properly proportioned argillaceous and calcareous materials and to which no addition greater than 3 per cent has been added subsequent to calcination.

Natural Cement.—The finely pulverized product resulting from a calcination of an argillaceous limestone at a temperature sufficient only to drive off the carbonic acid gas.

Sand Cement.—The finely pulverized product of intimate mixture in varying proportions (generally one-half of each) of silicious sand or other silicious material and Portland cement.

Puzzolan Cement.—The finely pulverized product of a mechanical mixture of volcanic ashes or basic blast furnace slag with powdered slaked lime.

When slag is used this is sometimes known as slag-cement.†

* The discussion occupied part of two sessions.

† There is a cement known in Germany as Eisen-Portland cement, which in England is frequently referred to as slag cement. This is formed by adding to the ground Portland cement clinker prepared as a true Portland cement, up to 30 per cent of pulverized slag. This should be distinguished from slag cement, as defined above.

III. AGGREGATE.

The inert material which in combination with cement and water comprises the essential ingredients of concrete.

Fine Aggregate.—Natural sand or stone screenings passing when dry a screen having holes ¼ in. in diameter.

Sand.—The finely divided material resulting from the natural disintegration of rock and graded as defined under "Fine Aggregate."

Stone Screenings.—The finely divided product formed by crushing a natural rock and of a size as defined under "Fine Aggregate."

Coarse Aggregate.—Inert material which is retained on a screen having holes ¼ in. in diameter and which is incorporated in the concrete in the mixing. The upper limit of its size depends on various conditions; in general, anything above 3 in. is known as a plum, and is added to concrete after mixing and during placing, thus forming rubble or cyclopean concrete.

Gravel.—Rounded rock particles graded from fine to coarse and occurring together with loam, clay, or other earthly substances in a natural bed or bank and meeting the above requirements for "coarse aggregate."

Crushed Stone.—The product resulting from crushing natural rock and meeting the above requirements for "coarse aggregate."

Crushed Slag.—Air-cooled basic blast furnace slag meeting the above requirements for "coarse aggregate."

Cinder.—The hard waste product of the combustion of anthracite coal.

Plums.—Stones of large size added to concrete after mixing and during placing.

Bank-Run Gravel.—The normal product of a gravel bank.

Run-of-Crusher.—The unscreened output of the stone crusher.

IV. REINFORCEMENT.

The metal (generally steel) embedded in concrete in proportionately small sections in such a manner that the two materials assist each other in taking stress.

Hard Steel.—Steel with a minimum ultimate tensile strength of 80,000 lb. per sq. in. and a minimum yield point of 50,000 lb. per sq. in.

Structural Steel.—Steel with a minimum ultimate tensile strength varying between 55,000 and 70,000 lb. per sq. in. and a minimum yield point of 33,000 lb. per sq. in.

V. GENERAL TERMS.

Mortar.—A mixture of sand and cement, with plaster, or with lime, wetted and mixed to the consistency of paste. Applied to concrete, the mixture of cement and fine aggregate, which lies in the voids of the coarse aggregate.

Grout.—The resulting mixture of cement and water or cement, sand and water, in fluid consistency.

Billet Steel Bars.—Bars rolled from new billets.

Rail Steel Bars.—Bars rolled from standard section T-rails.

Prof. Hatt.

PROF. HATT.—Probably the national organization that has had the most experience in framing definitions is the American Railway Engineering Association, which has adopted a rule that definitions shall not be discussed on the floor. This rule was adopted when experience had shown that results were not obtained by discussing definitions on the floor of a convention. I think the definitions are all right; they are apparently copied from definitions adopted by other societies.

(On motion of Mr. Wig, it was voted to adopt recommendation 2 in the committee's report. On another motion, it was voted to refer the definitions submitted by the Committee on Nomenclature to the Board of Directors for the latter to send out for letter ballot at its discretion.)

The President.

THE PRESIDENT.—I hope the Committee on Nomenclature will submit to the next convention a large engineering vocabulary, if not definitions. We ought to start an engineering vocabulary. For instance, a few days ago I encountered the word "tremie," and while I could infer from the context what was meant, there was not help to be found in any dictionary to which I had access. We should agree on something, so that all may understand what we mean when we use technical terms or, for that matter, common words to designate something in connection with concrete. Each member of the Institute should send to the committee all suggestions concerning the vocabulary of our specialty in which there seems to be any chance of a misunderstanding, and a list of all words which are used in our work to convey a different meaning from that attached to them in Webster's dictionary.

UNIT COSTS IN CONSTRUCTION.

By Sanford E. Thompson.*

The aim of this paper is to call attention to the necessity for more accurate estimates by architects, engineers and contractors, and to the possibility of reducing construction costs by a more thorough analysis and systematizing of the labor operations.

Open any technical paper giving construction items and we notice the enormous variation in bids received from contractors for the same job. At random I open a current engineering periodical and find the four bids on a certain job: $117,000, $126,000, $143,000 and $171,000. This does not represent an exceptional case, but is the general rule on work in which the cost of labor is relatively large.

Unquestionably the methods of management, especially on construction jobs, affect the cost to a very large extent, but it is hardly conceivable that differences of, say, 50 per cent in cost can be accounted for in this way. Assuming, as is generally the case, that the bidders really try for the job, there is evidently gross error in many cases in the estimating. The variations must be due to one of three causes: (1) inaccurate estimate of volumes or costs of materials; (2) inaccurate estimates of labor costs; or (3) inaccurate estimates of overhead costs and profits. Note that I have combined the items of overhead and profit because they are practically interdependent. The contractor, for example, who handles a job personally has small overhead expenses but requires larger profits to make up for the value of his own time than does a large contractor whose overhead in salaries and plant are large, but whose percentage of profit may be small and yet yield a large total income.

Material costs usually are figured without trouble. The variation in overhead by two different estimators may be large because many contractors do not properly charge or divide their overhead items, but this difference on any one job can hardly account for more than 10 per cent. The big variation, then, must be in the estimated labor cost. As a matter of fact, this is the item on which money is made or lost in contracting.

A contractor was asked recently why he did not bid on a certain reinforced concrete bridge. The reply was: "I have built mine." Construction companies handling reinforced concrete will tell you that a contractor does not begin to make money till he gets his third job, provided he stays in the game as long as this. The first job costs him more than his estimate; he bids low on the second job because he thinks he sees where he lost money on the first; on the third job he knows he has been a fool and adjusts his bid accordingly.

It must be the aim not simply to know how to do the work cheaply but to figure estimates more accurately, at the same time taking care to keep

* Consulting Engineer, Boston, Mass.

the cost of the figuring as low as possible. It is useless to get a job at a low figure and lose money on it. It is evident, further, that, having made the bid with care, the construction must be handled in such a way as to come within the limit set. This requires not simply a fair comparison with the monthly estimates, but the follow-up from day to day of quantities and wages so as to have a running check on the work. It takes but little more time in figuring an estimate where material costs are estimated in detail to go also into unit costs of labor. This of course is not simply a mathematical proposition, but requires a knowledge, based on first-hand experience, of actual costs, or else on records where conditions are stated in full.

In reinforced concrete construction, the greatest discrepancy lies in the cost of forms. It is here that the contractor and also the engineer are apt to be fooled, unless either they are well provided with unit costs or else have handled work previously of an identical nature. Suppose, for example, a builder has been accustomed to building forms for heavy concrete, such as core walls for dams, heavy foundation work and mass construction. He finds from his records that the cost of labor for such forms, not including overhead and profit, is 50 cents per cu. yd. He bids on a reinforced concrete building, and adds, say, 50 per cent, or say, 100 per cent to be on the safe side, and such a percentage guess as this is by no means uncommon in estimating, and figures $1.00 per cu. yd. for labor on forms. Instead of the $1.00, he may find the cost to be about $4.00 per cu. yd. and discovers further that the cubic yard basis is absolutely incorrect. He then adopts the square foot as the unit. This is better, but let us see how this may work out.

Suppose that his first job in building construction is a design calling for 250 lb. per sq. ft. of working load, with long spans, columns averaging 24 in. square, and beams and girders of similarly large size. The new job, we will say, has a load of 125 lb. per sq. ft. with columns averaging 15 in. and correspondingly smaller beams and girders. He realizes that he must not use the cubic yard unit for figuring his forms. If he did use this, he would find when he got through that the cost per cubic yard of his labor on the smaller size members was nearly double that on the other job. He uses then the square foot basis. On the building with heavy load, the labor cost of the column forms, using columns only, to be specific, was, we will say, 8.3 cents per sq. ft. We will suppose he uses this on the building with light loads. The estimate will be absolutely incorrect. It costs only a little less to make and to erect the smaller forms than the larger, although, on the other hand, enough less to throw the scale in the other direction if the cost is assumed equal, member for member. On the light-load building, as a matter of fact, the labor cost on the 15-in. columns is about one-quarter greater per square foot than the cost on the building with columns averaging 24 in. square, while the cost per member is 16 per cent less. Costs, based on accurate unit averages, checked by work on various jobs, show the following comparison of 15 and 24-in. column costs, each in a building of 12 ft. total story height and all other conditions the same.*

* From "Concrete Costs," by Taylor and Thompson, first edition, p. 631.

Size of Column.	Cost per Cu. Yd.	Cost per Sq. Ft.	Cost per Member.
24 in.	$4.50	$0.083	$8.00
15 in.	9.68	0.105	6.74
Per cent increase	115	27	16*

* Decrease.

You may say that this is an extreme case, but I am quoting almost exactly from an actual case of a contractor of large experience in building construction but with limited experience in reinforced concrete construction, who used, not the square-foot method, but the cubic-yard method of estimating in two buildings with heavy and light load. Fortunately for him, he built the heavy building first, and so simply lost the job on the second building by figuring way up in the air.

Much has been written of the inaccuracy of "cost data," and with perfect truth. On the other hand, if the unit costs are taken from personal experience or from records in which all the local conditions are fully stated and average values compiled, they are of immense value in estimating and in following up the work. Care must be taken, of course, to provide for indirect charges, such as foreman, sharpening tools, time on miscellaneous work, plant erection and contingencies. In addition to these are the charges for superintendence, contingencies chargeable to labor but not estimated as part of payroll, odd tools and appliances not carried to next job, liability insurance, etc.

To illustrate the variations in labor costs of different members in form costs of different members in form construction, the table below presents a few values selected from "Concrete Costs."†

Costs of this kind are useful not merely for contractor's estimate, but perhaps even more for comparing different designs by engineers and architects. Reference might be made, for example, to studies for the new Technology buildings, given in the paper presented by the author last year, in which it was found that for three different types of floor design a long span slab was the cheapest on account of the difference in the cost of form construction.

REDUCTIONS IN CONSTRUCTION COSTS.

Up to this point has been considered the substitution of accurate methods for guesswork in estimating. The more important question is how far the knowledge, such as has been referred to, can be utilized in reducing costs. Accurate cost keeping is of value in following up construction costs from day to day in showing up waste labor and in providing a mark for the attainment of superintendents and foremen. Unless cost knowledge is in the form of small units, such comparisons cannot be made satisfactorily.

To get the full benefit of a knowledge of unit costs, and in fact for this the knowledge must be even more thorough and include the unit times of performing the various operations, it must be utlized in the planning of the work in advance and in distributing materials and jobs; in selecting materials and

† For complete tables, see "Concrete Costs," by Taylor and Thompson, first edition, pp. 630–54.

methods which will result in lowest labor costs; in adapting the construction plant to the special conditions; and, carried to its ultimate end, in laying out jobs for the workmen and giving them a reward for accomplishment.

Such management as this involves the adoption of factory methods in construction. Already the need of this is being recognized, but only to a limited degree. The president of one of our large and most up-to-date construction companies advised me recently that on one job he made a saving of $10,000 by the adoption of methods involving systematic planning and routing.

Limitation of time forbids a more complete discussion of this most important problem. Full economy in construction, however, will only be attained as the builder discards the haphazard rule-of-thumb method and considers his job with a view to thorough analysis, planning functional methods and a complete study of details. By such methods as these will the labor of construction be brought to a more scientific basis and more nearly on a par with the material end of the work.

LABOR COSTS OF FORMS FOR COLUMNS, BEAMS, GIRDERS AND SLABS.

Costs include 10 per cent for foreman and 15 per cent for superintendence, contingencies, etc., but do not include profit or home office expense. Carpenter labor, 50 cents per hour; ordinary labor, 25 cents per hour. Material, 1-in. lumber.

Size, in.	Make Forms.	Place and Remove Form First Time.	Place and Remove Form After First Time.	Remake, Place and Remove Form.
12-FT. COLUMNS. LABOR COST PER MEMBER, IRON CLAMPS.				
8 x 8	$1.16	$4.68	$3.77	$5.53
16 x 16	1.46	5.45	4.40	6.16
24 x 24	1.80	6.20	5.14	6.86
36 x 36	2.61	7.64	6.33	8.14
20-FT. BEAMS. LABOR COST PER MEMBER, SIZE MEASURED BELOW SLAB.				
4 x 8	$0.92	$2.42	$1.97	$2.79
6 x 12	1.09	2.75	2.31	3.23
8 x 16	1.26	2.99	2.59	3.64
12 x 24	1.75	3.41	3.09	4.29
20-FT. GIRDERS. LABOR COST PER MEMBER, ONE INTERSECTING BEAM.				
8 x 16	$1.38	$3.27	$2.75	$4.31
12 x 24	1.82	3.86	3.20	5.02
LABOR COST OF SLAB FORMS* PER 100 SQ. FT. OF SLAB SURFACE.				
....	$0.81	$2.53	$1.90	$2.06

* Based on slab built two panels per bay.
For inexperienced builders, increase costs $33\frac{1}{3}$ per cent.
For special design, add 10 to 50 per cent to "Make Forms."
If no mill saw on job, add 50 per cent to "Make Forms."
If old lumber is used, add 75 to 100 per cent to "Make Forms."
For rectangular columns, select values for square columns having the larger dimension of the rectangle.
For wall columns, add 50 per cent to all except "Make Forms."

SOME SUGGESTIONS FOR THE DESIGN OF CONCRETE BUILDINGS.

By W. P. Anderson.*

Reinforced concrete buildings are often uneconomically designed. This is sometimes due to unnecessary restrictions in specifications; sometimes also largely to improper proportioning of various members, and sometimes to neglect to consider the effect of the design on the cost of the form work. Records of detailed costs of reinforced concrete buildings made by our company in the last fourteen years, together with tests which we have made on actual buildings and in our laboratories, have given us data not available to all designers and, therefore, some of these data and the conclusions we have drawn therefrom may be of interest.

Floor Slabs.

Thickness.—Unnecessary restrictions sometimes occur in specifications in reference to floor slabs over beam and girder construction.

Sometimes a minimum thickness restriction on the floor slab is specified, due probably to the belief that if the slab is too thin, heavy falling objects may punch through the floor. We have built many buildings where 3-in. slabs were used and have had numerous instances where heavy timbers have fallen on end and other material has dropped on these floors without damaging the slabs. These experiences have convinced me that if the reinforcement is close enough together, the slabs can be as thin as the stress requirements permit. The resistance of reinforced concrete to sudden jars was remarkably shown during the erection of a warehouse for the Fireproof Storage Company of Cincinnati. A piece of artificial stone weighing 1150 lb. fell 40 ft. onto a concrete slab and did not damage the beams or slab. When this occurred the form panels had not been removed, although the supports under them had. In slab-spanning in one direction, two and one-half times the slab thickness is a satisfactory maximum spacing.

Steel at Supports.—Some designers believe that it is necessary in continuous floor slabs to have enough steel at the supports to take a moment of $WL/12$. If, however, the steel at the center is sufficient to take this moment, I do not believe it necessary in the slabs for beam and girder construction to have this much steel taking tension at the support.

We have made a number of laboratory tests and severe tests in actual buildings and some of these caused high moments in the floor slab at the supports due to loading two adjacent panels, the other panels being without live load. A test was made eight or nine years ago at the Fireproof Storage Warehouse by our company, to satisfy the Engineers of the Louisville and

* President, The Ferro Concrete Construction Co., Cincinnati, O.

Nashville Railroad as to the strength of our design. The floor was designed for 200 lb. per sq. ft. It was intended to make a destruction test, but our object was accomplished when we had applied a live load of 850 lb. per sq. ft. over two adjacent panels. Although when the floor was constructed we had no idea of making a test of this kind, no damage whatever to the construction could be found.

The clear span was 7 ft. 6 in., slab thickness $4\frac{1}{4}$ in. plus $\frac{3}{4}$-in. finish, with $\frac{3}{8}$-in. square twisted bars 7 in. on center, half of the bars bent up at the supports. The distance from the center of the bar to the extreme fiber at the support was 3.75 in. and the area of tension steel at the support in 1 ft. width of slab was 0.12 sq. in.

Then, if the moment at the support was $WL/12$, this steel would have been stressed to 125,000 lb. per sq. in., assuming the effective depth at 0.91 times the distance from the extreme compression fiber to the center of the steel. I have used 0.91, as the neutral axis will be higher than usual on account of the small percentage of steel. However, I do not believe the steel was stressed so high.

Another test of 1200 lb. per sq. ft. on a floor designed for a live load of 300 lb. per sq. ft. was made at a warehouse which we erected for the Louisville and Nashville Railroad at Atlanta, Ga., the structural design being made by us. One loading was made where the two adjacent panels touch over a beam framing into a girder, the other loading being on adjacent panels touching over a beam framing into columns.

The clear span here was 5 ft. $3\frac{3}{4}$ in.; the slab thickness $4\frac{1}{4}$ in. plus $\frac{1}{2}$-in. finish, with $\frac{3}{8}$-in. square twisted bars $8\frac{1}{2}$ in. on center, half of the bars bent up at the supports. This low percentage of steel was probably due to the fact that the slab spans in some other parts of this floor were longer than they were where the test was made, but the same depth of slab was used throughout, the longer span governing the depth. The distance of the center of the bar from the extreme fiber was $3\frac{1}{2}$ in. at the support, and the area of the tension steel at the support, in 1 ft. width of slab, was 0.099 sq. in. Thus, if the moment at the support under this load was $WL/12$, the stress on this steel would be about 114,000 lb. per sq. in., an improbable assumption.

When the steel at the support is stressed beyond its elastic limit, as may have been the case under the extremely unusual conditions in these tests, the steel at the center at once begins to take a much greater increment of stress with increasing loads and relieves the stress at the support. Of course it is as cheap and possibly a little bit cheaper to bend up all the bars at the support in a floor slab, but I feel that where half the bars are bent up, the steel is probably better placed for the general use to which the building is subjected, and avoids the possible danger of not providing enough steel at the bottom near the supports.

When girders cause compression in a floor slab, this compression reduces the tension caused by the bending of the slab itself, and thus may reduce the stress on the tensional floor steel.

Economy in Design.—Sometimes a saving can be made in designing floor slabs by not stressing the steel to its limit, thus reducing the thickness of the

slab, or by increasing the slab thickness and reducing the amount of steel, thus stressing the steel but not the concrete to the limit. This is determined by the relative cost of concrete and steel and the percentage of effect on the members effected by the increase or decrease in dead load. Often, however, it pays to thus increase or decrease the slab thickness, even though this increases the combined cost of concrete and steel, as it simplifies the form work. I have seen designs where several different slab thicknesses, due to different span lengths, occur in adjacent spans, making a number of different types of beam forms necessary, when it would have been considerably cheaper to have used one thickness for all spans.

BEAMS, GIRDERS AND BRACKETS.

Economical Forms.—In the design of beams, girders, brackets or projections, one should consider the effect on the form work. I have seen the dimensions of beams, similarly placed, changed from floor to floor, or floor to roof, when the live load changed, although the size of the building made it advisable to re-use the same form work. This meant that the form work had to be re-made, which practically always causes a much greater expense than the saving of material due to change in the dimension of the members. I have also seen projections from lintels, to carry brick or terra cotta, made in the center of the side of the lintel instead of at the top, thus making the form work for the side of the lintel more expensive.

Shear Tests.—A number of years ago our company made a number of tests on shear, as shown in Figs. 1 to 4, and somewhat similar tests were made at the University of Cincinnati within the last year. In our tests values of over 1000 lb. per sq. in. were obtained. All the beams but one had concentrated loading, and the one which was uniformly loaded we were unable to break with the available loading material, although the end shear equaled 1238 lb. per sq. in. The first noticeable hair cracks appeared at shear stresses ranging from 182 to 339 lb. per sq. in., regardless of the reinforcement, but the ultimate shear value depended on the shear reinforcement and the methods of loading.

Tension.—Fig. 5 shows a simple beam with two loads L applied at distances from the supports R equal to the effective depth. At the point of application of L this force can be resolved into two compressive forces as indicated. The diagonal component resolves itself at the support into a downward reaction R and a horizontal force. This horizontal force must be transmitted to the tension bar, which can be done by a stirrup, as shown on the right-hand side, or by extending the tension bar beyond the support sufficiently to obtain the necessary grip as shown on the left.

Fig. 6 represents a truss made of six separate pieces A, B, B, C, C and D, the only rigid connection being at the points O beyond the supports, and shows how the grip can act beyond the support, indicating how the bar running beyond the support in Fig. 5 assists in strengthening the beam. This will only happen, however, in simple beams, as in continuous beams the horizontal component at this point is in compression and does not have to be

FIG. 1.

FIG. 2.

FIG. 3.

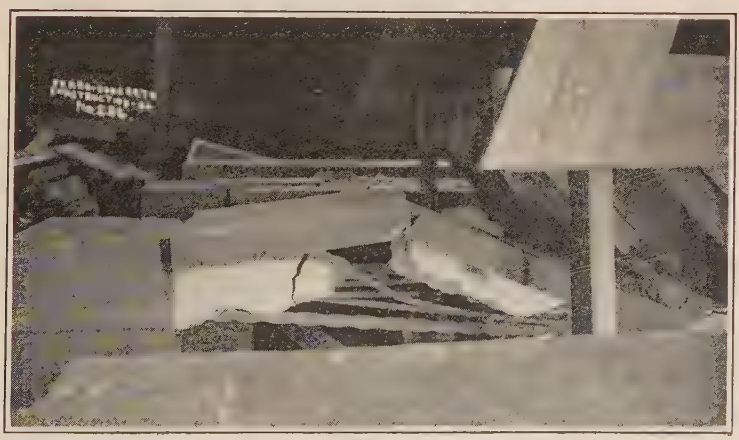

FIG. 4.

transmitted to the tension bar. It is advisable, however, in continuous beams, to run some of the tension bars at the bottom of the beams 8 or 10 in. into the support, as this helps the compression at this point and also insures having steel to take tension at the bottom for such loading as causes the point of contraflecture to approach the support. When the necessary grip cannot be obtained in a bar without running it beyond the point of contraflecture, this should not be done to obtain the grip, but a smaller size bar should be used.

Figuring Spans.—In figuring spans for concrete members, in cases where the moment of inertia of the member is greatly increased at the support, as is the case where it frames into other concrete members, I believe it is more conservative to use the clear span and not a distance greater than the clear span. Where the moment of inertia at the support is several times greater than the rest of the beam, and assuming a uniform moment of inertia for

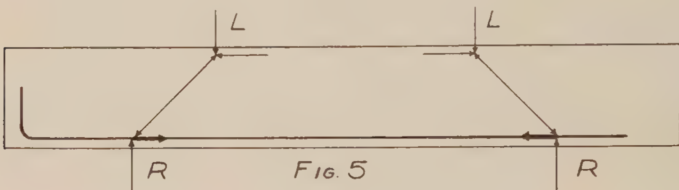

FIG. 5

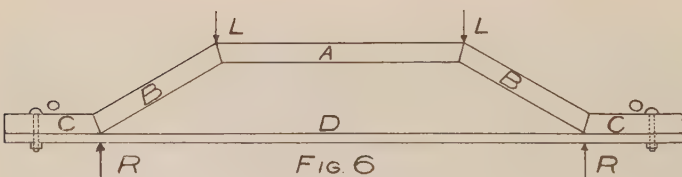

FIG. 6

the beam between the supports, the positive moment at the center for fixed beams is $WL/24$, no matter what the extent of the support, L being the clear span. If a greater value than the clear span is used for L and the point of contraflecture is determined from this value, it will be obtained nearer the support than is actually the case, and there is a chance that the negative moments may not be provided for as far from the supports as they should be. Where enough steel is used at the center of a continuous beam to provide a moment of $WL/12$ and the beam is so loaded that theoretically it would, assuming a uniform moment of inertia, have twice as much or more moment at the support as in the center, I believe it is unnecessary to provide for a moment of $WL/12$ at the support. This is partly due to the fact that the beam is not of a uniform moment of inertia, but more largely to the fact that when the stress at the supports passes a certain point and the concrete

and steel become highly stressed, the excess concrete and steel at the center relieves the high stress at the supports.

Wenalden Warehouse Test.—Fig. 7 shows a test made under the direction of Prof. A. N. Talbot at the Wenalden Warehouse in Chicago, which we erected for Carson, Pirie & Scott. Here the beams had only half as much steel at the support as at the center. A complete report of this test can be found on page 61, Vol. VIII of the *Proceedings* of this Institute. The report of these tests shows that the steel at the supports was not stressed nearly as high as a moment of $WL/12$ would produce, although the recorded stresses of concrete at supports was about what this assumption of moment would give. It will be noted in Table II, page 76, of that report, that when a single panel was tested the stress in the steel at point 114, at the center of the

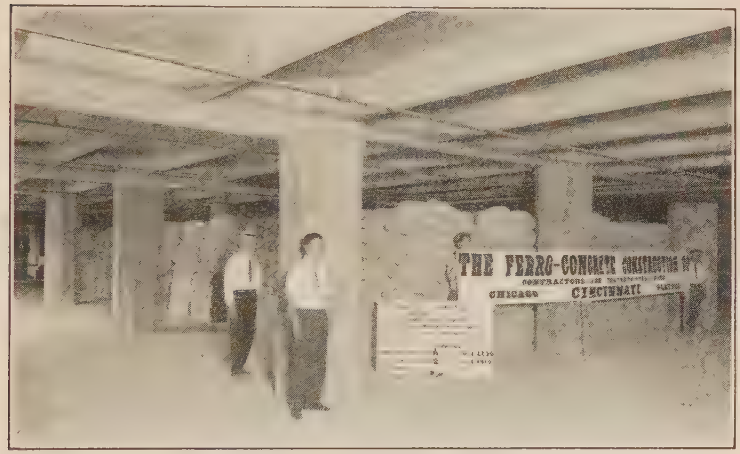

FIG. 7.

beam, was 6000, while when this panel and the two adjacent panels were loaded, the stress was increased at this point to 11,000 lb. Such action as I suggest could account for this increase, where a decreased stress should be expected from the additional load.

Stresses at Connections.—We are at present making some tests in order to determine what higher stress can be used at the point where a concrete beam frames into a larger mass of concrete.

The tests already made cover beams 3 in. wide, 8 in. deep and 4 ft. 6 in. long, some having a uniform section throughout as shown in beams 1A and 1C, Figs. 8 and 9, and some having at the center of the beam an enlarged block of concrete, 11 x 11 in., projecting 4 in. below the compression side of the beam, as shown in beams 2A, 2B, 3A and 3B, Figs. 10 to 13. The reinforcement consisted of four $\frac{1}{4}$-in. Havemeyer bars, bent as shown in Figs. 9,

14 and 15, the concrete having been broken away from the end that failed, so that the steel could be shown. The two No. 1 beams were alike, as were also the two No. 2 and the two No. 3.

Cracks occurring on the beams during testing were marked with lead pencil so that they would show up, and the load which caused these hair cracks was noted on the beams. The beams were concreted January 29,

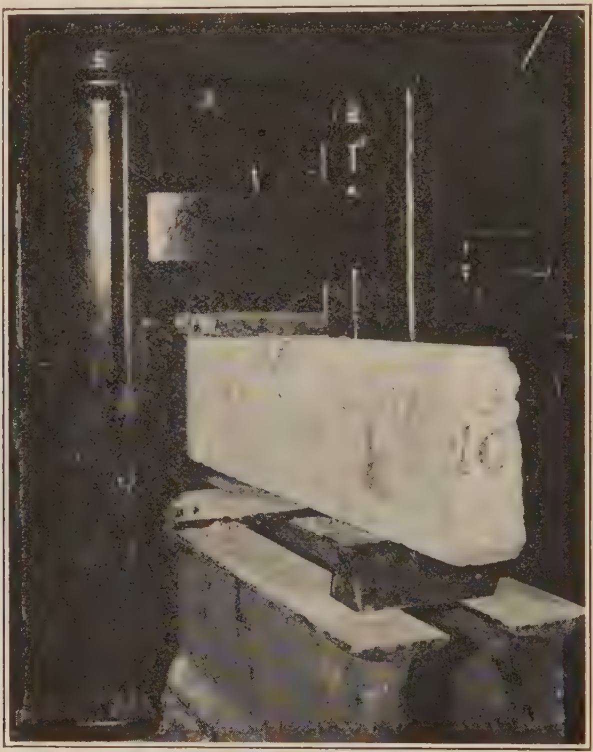

FIG. 8.

1916, and were tested February 11, 1916, being two weeks old. The average strength of four 5-in. cubes made from the same batch of concrete and tested on the same day as the beams, was 1382 lb. per sq. in. The central load was applied in all cases by bearing against a $1\frac{1}{4}$-in. plate 11 in. long, and the end loads were on $\frac{1}{8}$ x $1\frac{1}{2}$-in. plates resting on small round supports placed 15 in. from the edge of the center plate, nearest the end support, the moment in inch-pounds thus being 0.5 $L \times 15$, L being the test load.

The beams without an increased section failed at the edge of the central support, but those with the enlarged section failed at about an inch out

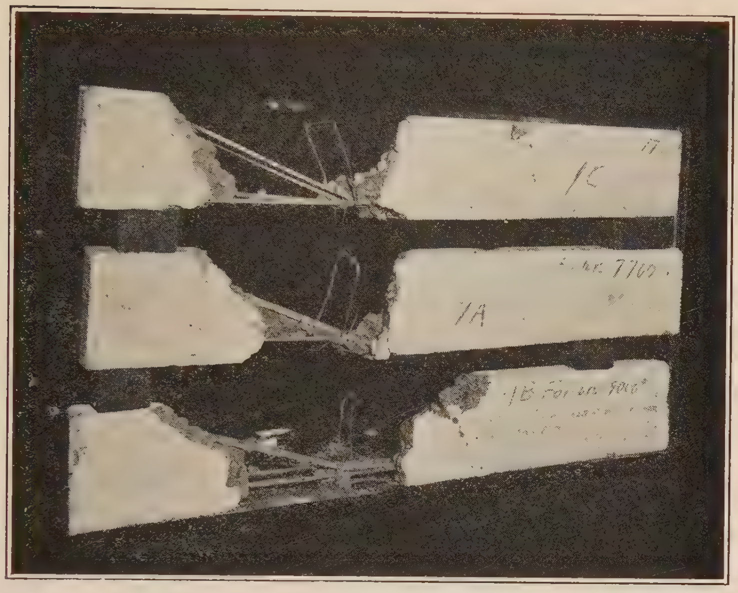

FIG. 9.

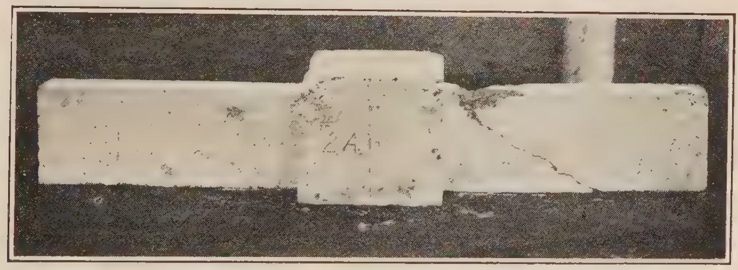

FIG. 10.

FIG. 11.

FIG. 12.

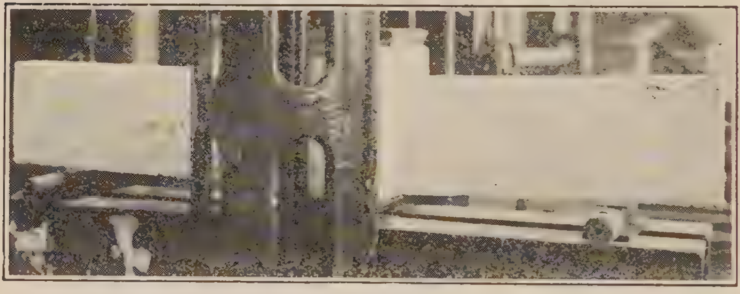

FIG. 13.

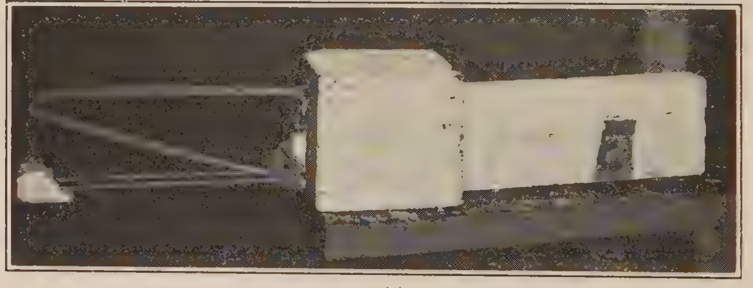

FIG. 14.

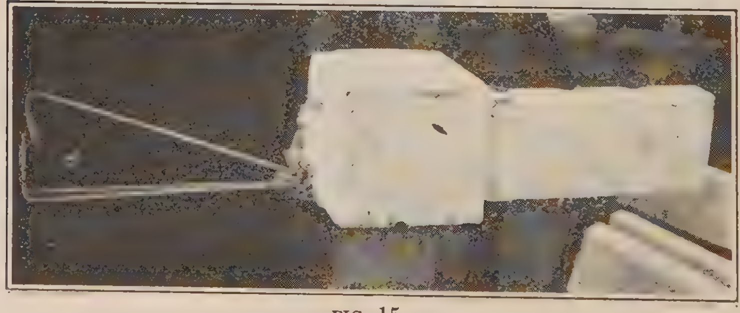

FIG. 15.

from the central support. It was my intention to have four different kinds of beams: first, with uniform sections throughout and no steel at the compression side at the central support; second, like the first but with compression reinforcements at the compression side of the central support with wire ties encircling the compression and tension steel near the support; third, like the first as to reinforcement, but with enlarged section; fourth, like the second

Beam No.	Loading and Arrangement of Steel	Ave. Clear Dis Steel to Face of Concrete	L Load at Failure
1A	Beams #1	1 3/16"	7700 lbs
1B	Beams #1	1 3/8"	
1C	Beams #1	1 9/64"	7000 lbs
2A	Beams #2 & #3	56/64"	8960 lbs
2B	Beams #2 & #3	1 17/64"	8640 lbs
3A	Beams #2	2 1/32"	11,330 lbs
3B	Beams #3	56/64"	10,230 lbs

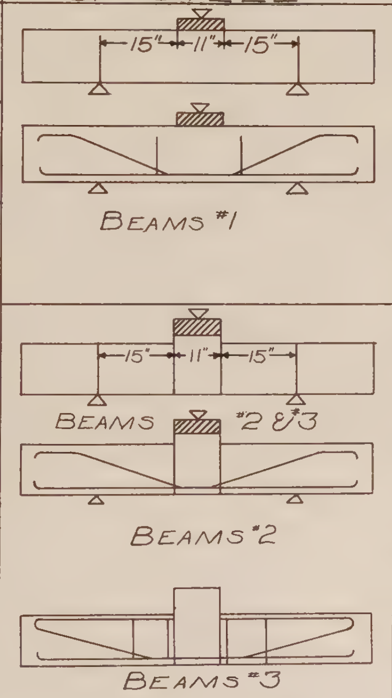

Table I.

as to reinforcement, but with enlarged section. I was not present, however, when they were made, and while the third and fourth cases were as intended, the first two cases were made alike without the compression bar, but with one wire tie just outside the central support.

The results are shown in Table I and the beams with the increased section show higher values for the compression in the extreme fiber than those

of uniform section, while the compression steel helped materially to increase the strength. This indicates that when a concrete member frames into a larger concrete member a greater compressive stress can be allowed than is used on the extreme fiber when this condition does not occur.

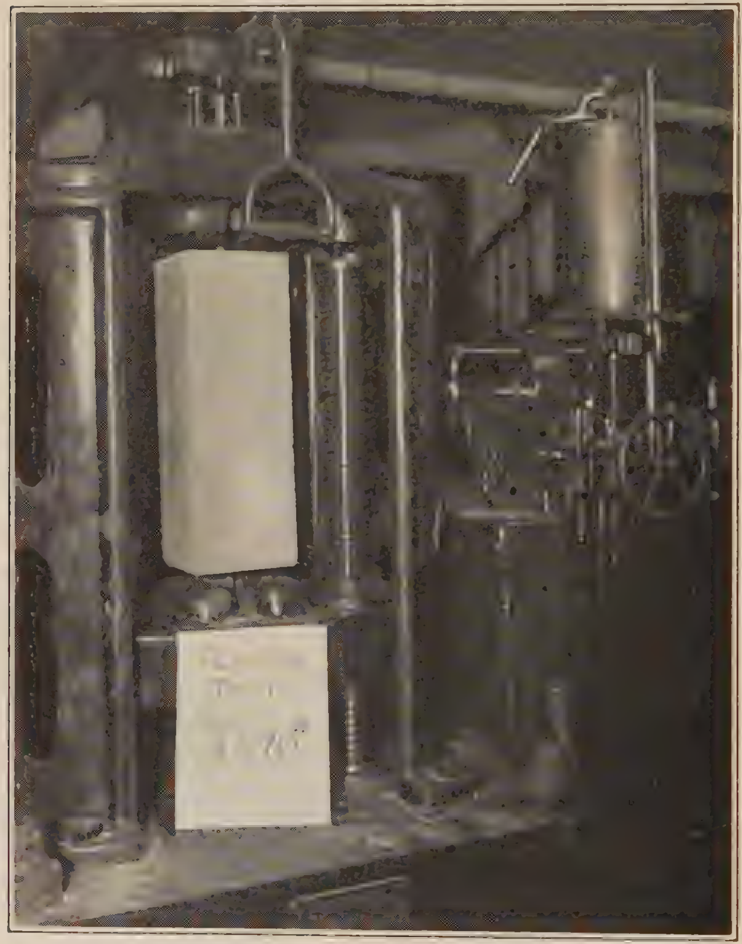

FIG. 15.

Tests of Bond.—We are also making some tests at the present time to determine whether or not the same value should hold for the grip of bars in compression as used in tension. These consist of test pieces as shown in Fig. 16. The test pieces were concreted January 29, 1916, from the same

batch from which the beam tests were made on that date and were tested February 11, 1916. The concrete blocks were 18 in. long and had a longitudinal bar near each corner to prevent the tensional failure of the concrete. The center bars were $\frac{3}{8}$-in. Havemeyer bars projecting 8 in. into the block from either end, being in line and separated 2 in. Three pieces were tested by pulling on the center bars, causing a tensional grip failure, which occurred as follows:

Piece 1: first slip, 2520 lb.; failure..............	4570 lb.
Piece 2: first slip, 3690 lb.; failure..............	4860 lb.
Piece 3: did not slip; failure....................	4700 lb.
Average, 392.5 lb. per sq. in., or.................	4710 lb.

Average 4710 lb., or 392½ lb. per sq. in. of bar contact.

Three other pieces were tested by compression, the projecting bars being cut off and the ends filed, leaving a projection of ¼ to ½ in. The pieces were then tested by compressing against the projecting bars, with results as follows:

Piece 4: bars projected about $\frac{7}{16}$ in.; failed at 6895 lb.; one of the projecting bars bending.

Piece 5: bars projecting about ¼ in.; failure at 7800 lb.

Piece 6: bars projecting ½ in. one end, ¼ in. at other end. At 8000 lb. the projection on the ¼ in. end started to decrease, but held the load to 10,000 lb.

The average of the three tests was 8230 lb.

Concrete cubes made of the same concrete and tested on the same day showed an average strength of 1382 lb. per sq. in. If we double this value and multiply by the area of the end of the $\frac{3}{8}$-in. bar, we will get 387 lb. as the increase from the end bearing. The total increased strength of the compression tests over the tension tests was, however, much greater than this, and, while some of this additional increase may be due to the fact that the concrete at the end of the bar is capable of holding over twice the compressive strength shown by the cube tests, a large part of it is undoubtedly due to the fact that when the rod is in tension its section decreases as the stress increases, thus pulling away from the concrete, while when the bar is compressed its section enlarges with increased stress, thus binding tighter with the concrete. These tests indicate that the distance required to transmit a given stress from a reinforcing bar to the surrounding concrete is much less when the bar is in compression than when it is in tension, but I am not prepared to give definite rules until more tests have been made. The testing on February 11, 1916, was done at the University of Cincinnati, to whom I am indebted for the use of their testing machine and assistance given.

Brackets.—The effect of brackets on beams is to reduce the central moment and increase the moment at the supports, throwing the points of contraflecture nearer the center of the beam. I would recommend in cases where brackets are used that the upper bend in the bent-up bars should start a distance out from the support about equal to half the depth of the

beam plus the depth of the bracket, and that some of the bars in the bottom of the beam should not be bent up, but should run well into the bracket.

In designing flat slabs, the usual formulas assume a fixed ratio between the size of the column head and the panel, and they are frequently based on the column head, being 0.225 times the size of the panel. As a matter of fact, a number of column head forms are in use where the size cannot be altered, and where these are used it is not always practicable to keep the ratio assumed in the formulas referred to above. If the ratio is less than that assumed, the slab will not be as strong as required and if the ratio is greater the slab will be unnecessarily strong. I would suggest that a formula be used for flat slabs which will be applicable to various ratios of size of column head to panel. This can be done by increasing the moment factor and using for the span length in the formula, instead of the center to center span, the distance between the center of pressure of the edge of the supports. Thus, in a round capital support, the reduction on that end would be about two-thirds the radius of the capital, and in a straight support, like a lintel, without column capital or bracket, the edge between the slab and lintel should be taken.

FORMS FOR CONCRETE WORK.

By R. A. Sherwin.*

The paper on the "Strength of Concrete Forms," presented by Mr. H. S. Taft before the 1915 convention of the American Concrete Institute (*Journal*, February, 1915), must have been received by its members with a great deal of interest. It showed considerable time and thought spent in its preparation. I propose to discuss some of the points mentioned in that paper and endeavor to stimulate further interest and study by describing the methods used in handling form work by the company with which the writer is connected.

Numerous cost analysis diagrams prepared in our office from the actual costs of average reinforced concrete buildings of the industrial type, not taking into account the cost of any equipment, show that the labor on forms averages about one-third the labor cost, which is about 35 per cent of the total cost of the building, and the lumber about one-tenth of the total material cost, including sub-contracts. The design and erection of forms is, therefore, the most important single item in concrete building construction and one upon which much thought and study can be well spent. Exchange of ideas by those familiar with concrete forms will be profitable to all concerned.

I do not propose to go into the mathematics of form design as extensively as Mr. Taft did in his paper. I do propose to show the relation between the important features of form work and show how we have successfully reduced the cost of forms by a careful study of their design, buying material and field work. I shall introduce some tables and figures to show how we apply the theory of mechanics to a problem which has too many variables to admit of any rigid solution. These tables have been found practical for use and have given safe results.

I do not agree with the following extract from an article on forms by Mr. Jerome Cochrane in the June, 1915, number of *Concrete-Cement Age:* "Too much stress cannot be placed upon the importance of having an ample number of shores under the forms, which should be regulated by the judgment of the foreman or boss carpenter." This is a question of design, and a drawing should be given the foreman which will give him all the important information in regard to supporting the forms.

The same author in the March, 1915, issue of the same paper writes in part as follows: "Plans and details for forms, if required to be furnished by the contractor, should be submitted to the engineer for his approval before starting the work, but such approval should not release the contractor from any responsibility, etc." I do not believe this is the correct position to take in this matter. The engineer should be interested in results only. The finished building must be as he specifies. Since he holds the contractor responsible for results, the methods of doing the work should be left to the

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contractor entirely. The engineer has his work in designing the structure. The contractor ought to know more about designing forms. It is his business to know how to build.

General Problem.—Economical forms must be designed as light and with as few sticks as possible to give the necessary strength and stiffness. You will note I say "necessary," for this is the point to keep in mind. We must remember on the one hand that human lives and the cost of failure, both in money and reputation, are dependent upon the design, and on the other hand that form work is a temporary structure fully loaded for a short time only and a low factor of safety is therefore allowable.

The main question, then, is one of strength with enough stiffness to prevent any appreciable sags in the finished concrete. Any slight deflection that might occur in the timbers themselves is taken care of in the camber always given horizontal members in erection.

The strength of the floor timbers should be designed for the full live and dead load, but any serious deflection will be caused only by the dead weight of the wet concrete. The full live and dead load can act together for a very short space of time only. Therefore I think that limiting the centering by such small deflections as Mr. Taft proposed and using full live and dead load is questionable practice. One place where deflection should be carefully guarded against is at the window head. At this point a slight deflection will cause much trouble and expense in setting the sash.

The principal cause of settlement of form work is crushing of the soft spruce or pine lumber perpendicular to the grain in the girt over the post, and also in the adjustment of the wedges under the posts as the full load is applied.

In this connection Professor Johnson, on page 468 of his "Materials of Construction," says: "Since timber is very weak in crushing across the grain, as compared to crushing endwise, this is found to be one of the most common methods of failure in practice. It is common to rest a timber column on a sill of the same wood and to design the column for its maximum working load, paying no attention to the utter inability of the sill to carry this load without crushing. Many failures of timber structures are due to this cause alone." An average safe value to use in form design for this crushing stress, as determined by numerous tests, is 400 lb. per sq. in. for spruce. Considering this stress, a 4 x 4-in. spruce post under a 4-in. wide girt is good for 6400 lb. This is considerably less than the value of 16,800 lb. shown in Mr. Taft's table for an 8-ft. strut.

The trouble due to wedges can be almost entirely eliminated by using large hardwood wedges. If the posts are cut square at the ends to give an even bearing and heavy wedges used to bring the posts to grade, no settlement will occur.

Posts are usually placed on a plank sill. When these sills are laid on the ground great care is needed in order to avoid settlement because of deflection in the plank when a hollow place comes under the post. This often occurs when sills are placed on frozen ground, and salamanders thaw out portions of the surface and cause soft places under the sills. When light sills are placed

on a rough concrete floor deflection is liable to occur unless care is taken to give the sills an even bearing.

For these reasons I believe that limiting the floor timbers to very small

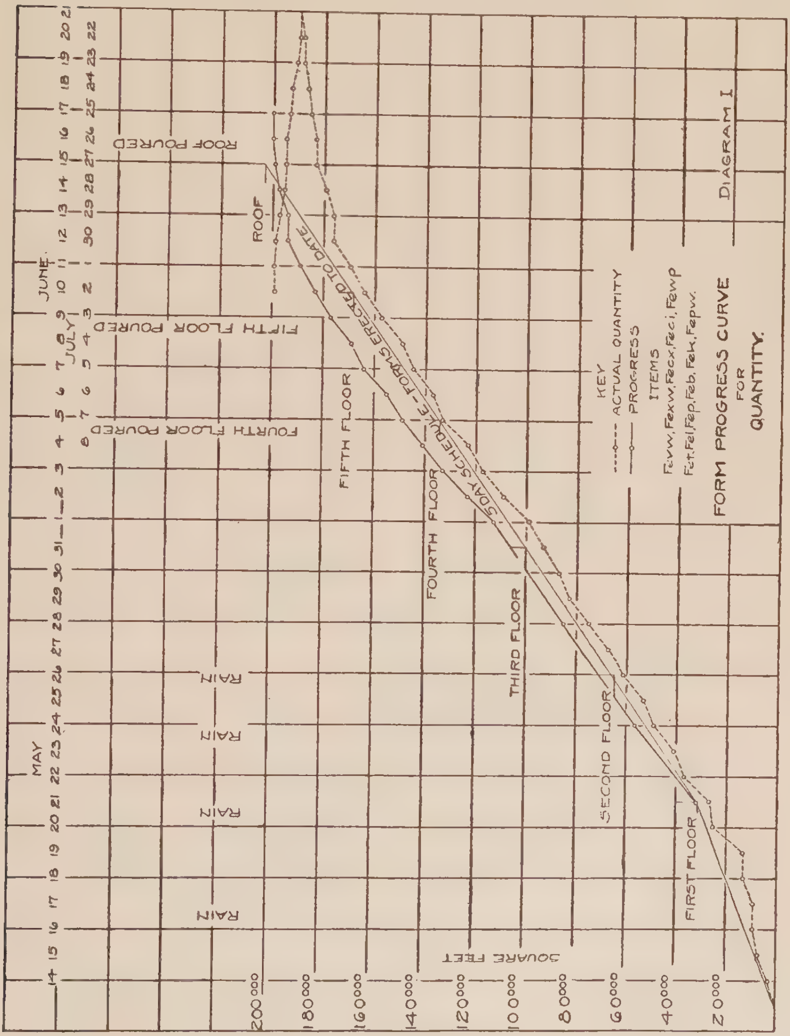


FIG. 1.—A TYPICAL PROGRESS CURVE FOR REINFORCED CONCRETE BUILDINGS.

deflections and at the same time using high loads on the posts is not consistent, and surely not economical.

Design.—The first thing to be done in preparing a form design for a concrete structure is to make a careful analysis of the plans submitted by the

engineer. This should be done as soon as the preliminary plans are available, for many details which seriously affect the progress and cost of the forms may be brought to the engineer's attention and changes made for the benefit of the owner. These points are often overlooked by the engineer, who does not always look at the structure from the construction point of view.

Chief among these points is that of story heights. It is desirable that these heights be kept uniform throughout the building so that the column forms can be reused from story to story without either cutting off or splicing. It is better to have the high stories at the bottom, as cutting off forms is much cheaper than splicing them.

Another important point is that of joints. I refer only to construction joints. These should be planned and reinforcement detailed so that the forms can be erected in the simplest possible manner. These joints should, of course, come at a natural stop for a day's work and be located so that they will least affect the appearance of the finished structure. The position and detail of any expansion joints should be determined early, as they will affect the form design and probably require special study.

An example of a detail to be considered in connection with the question of joints is the important one of making the dowels for the column rods long enough so that the necessary lap will come above the point where the column forms will actually start. It is not uncommon, for example, to find a wall beam, which comes above the slab, detailed so that it must be cast with the floor, but at the same time to find the lap for the column rods shown above the top of the slab instead of above the top of the wall beam, as it should be.

It is economical to cast a curtain wall beam, used sometimes in mushroom construction, as a later operation. The forms can be made and erected more cheaply. This can easily be arranged for by providing pipe sleeves through the column to take the negative rods and a slot in the column as a seat for the beam.

The kind of windows must be decided upon at once so that the necessary groove strip can be scheduled and attached to the column sides and beam bottoms at the "making-up" benches.

The floor finish will affect the form design. Questions as to the thickness of wood floor, or if granolithic, whether it is to be placed with the slab or as a separate operation, must be decided before the form design can proceed.

It is usual in the writer's work that excavation for the building begins at the same time the plans are started, or when the latter are in a very preliminary state. It is necessary, then, for the person in charge of the form design to be familiar with the structural design so that points such as have been noted may be settled, the lumber bought and the forms made up by the time the excavation is completed. When a complete set of plans are available the problem is easier.

The next step in the design is the preparation, by the cost department, of a progress diagram similar to Fig. 1. A description of the method of making the diagram is not within the scope of this paper. The limiting factor, however, is usually the date when the owner wishes the building turned over to him as part of his equipment. This diagram gives the dates on which the

various floors must be completely formed in order that there will be time for the necessary "follow-up" work. These dates will determine how many floors of new forms are necessary and when the design must be complete in order to allow sufficient time for making-up and erecting. The number of men necessary to turn out the work in the drafting room in this time can also be estimated.

With this preliminary information at hand, a conference is now held between the one in charge of the form design and the job and traveling superintendents. The latter is constantly in touch with the lumber market and is able to tell when the design must be governed by any unusual conditions. The sizes and lengths of lumber stocked in different localities vary considerably, and it is always desirable to make up the design of sizes that can be obtained at the lowest cost, in the required time and in the necessary quantity. Delays because of non-delivery of the lumber are costly.

Another point to be settled is the part of the job where construction will begin. This matter is usually governed by field conditions and has to be decided by the superintendent in charge after his study at the site. The movement of the forms throughout the job is dependent upon where erection is to begin and, therefore, must be known early.

The size, kind and location of the concrete plant has some effect upon the form design and should be discussed at this conference. The capacity of the plant will determine a day's concrete work and will affect the location of construction joints. The speed of the work is also sometimes dependent upon the concrete plant.

At this time the general scheme of the form design can be discussed and any suggestions from the experience of the superintendents may be incorporated. If the job superintendent and his carpenter foreman are in sympathy with the scheme a better job will result from this team work.

In calculating the sizes of form members, we do not use a standard set of loadings, formulas, etc., for all cases, but treat each one separately, according to conditions. In general we use the following values:

Dead weight of concrete	150 lb. per cu. ft.
Construction live load on floor forms	75 lb. per sq. ft.
Pressure of concrete on columns and walls per foot of depth	140 lb. per sq. ft.
Coefficient of elasticity for spruce or equal	1,200,000
Extreme fiber stress in spruce or equal:	
For timbers	1200 lb. per sq. in.
For column yokes	1800 lb. per sq. in.
Horizontal shear for spruce or equal	200 lb. per sq. in.
Crushing perpendicular to grain in spruce or equal	400 lb. per sq. in.

The proper value for the equivalent hydraulic pressure of concrete to use in form design has been variously estimated all the way from 75 lb. per sq. ft. per ft. of depth to twice that amount. The value of 140 lb., noted above, will give safe results and is based on actual experiments under job conditions.

One of these experiments was made on two columns and has already been reported in *Engineering News.* These columns were filled in a short space of time and the several mercury gages gave consistent readings of about 140 lb. per sq. ft., water equivalent.

With a slightly different arrangement of apparatus, we have recently made three tests of concrete pressure on wall forms. Two of these were in a

TABLE I.—SPACINGS AND SPANS FOR JOISTS.

Slab Thickness, in. / Weight, lb. per sq. ft.[1]		3.0 / 112.5	4.0 / 125.0	5.0 / 137.5	6.0 / 150.0	7.0 / 162.5	8.0 / 175.0	9.0 / 187.5	10.0 / 200.0	11.0 / 212.5	12.0 / 225.0
Size of Joists, in.	Spacing of Joists, in.	Span of Joists in Feet.									
2 x 6[2]	18	7.2*	6.9*	6.5*	6.3	6.0	5.8	5.6	5.4	5.3	5.1
"	20	6.9*	6.5*	6.2	5.9	5.7	5.5	5.3	5.1	5.0	4.8
"	22	6.5*	6.2	5.9	5.7	5.4	5.2	5.0	4.9	4.8	4.6
"	24	6.3	5.9	5.7	5.4	5.2	5.0	4.8	4.7	4.6	4.4
"	26	6.0	5.7	5.4	5.2	5.0	4.8	4.7	4.5	4.4	4.2
"	28	5.8	5.5	5.2	5.0	4.8	4.7	4.5	4.4	4.2	4.1
"	30	5.6	5.3	5.1	4.9	4.7	4.5	4.3	4.2	4.1	4.0
2 x 8[3]	18	9.7*	9.2*	8.8	8.4	8.1	7.8	7.6	7.3	7.1	6.9
"	20	9.2*	8.8*	8.3	8.0	7.7	7.4	7.2	6.9	6.7	6.5
"	22	8.9*	8.4	8.0	7.6	7.3	7.1	6.8	6.6	6.4	6.2
"	24	8.4	8.0	7.7	7.3	7.1	6.8	6.5	6.3	6.1	6.0
"	26	8.1	7.7	7.3	7.0	6.7	6.5	6.3	6.1	5.9	5.6
"	28	7.8	7.4	7.1	6.8	6.5	6.3	6.1	5.9	5.7	5.5
"	30	7.5	7.2	6.8	6.5	6.3	6.1	5.9	5.7	5.5	5.3
2 x 10[4]	18	12.2*	11.6*	11.0*	10.6	10.2	9.8	9.5	9.2	8.9	8.7
"	20	11.6*	11.0*	10.5	10.0	9.7	9.3	9.0	8.7	8.4	8.2
"	22	11.1*	10.5	10.0	9.6	9.2	8.9	8.6	8.3	8.0	7.8
"	24	10.6	10.0	9.6	9.2	8.8	8.5	8.2	8.0	7.7	7.5
"	26	10.5	9.6	9.2	8.8	8.5	8.2	7.9	7.6	7.4	7.2
"	28	9.8	9.3	8.9	8.5	8.2	7.9	7.6	7.4	7.1	6.9
"	30	9.5	9.0	8.6	8.2	7.9	7.6	7.3	7.1	6.9	6.7

[1] Weight includes dead load of slab plus 75 lb. per sq. ft. live load.

[2] Span which will develop 1200 lb. per sq. in. fiber stress and deflection of

$$\frac{\text{Span (in.)}}{360} = \frac{48\times 5.75\times 1{,}200{,}000}{5\times 360\times 1200\times 2} = 77 \text{ in. or } 6.4 \text{ ft.}$$

$$Mr = 2\times 5.75^2\times 1200/6 = 13{,}200 \text{ in.-lb.}$$

[3] Span which will develop 1200 lb. per sq. in. fiber stress and deflection of

$$\frac{\text{Span (in.)}}{360} = \frac{48\times 7.75\times 1{,}200{,}000}{5\times 360\times 1200\times 2} = 103 \text{ in. or } 8.6 \text{ ft.}$$

$$Mr = 2\times 7.75^2\times 1200/6 = 24{,}000 \text{ in.-lb.}$$

[4] Span which will develop 1200 lb. per sq. in. fiber stress and deflection of

$$\frac{\text{Span (in.)}}{360} = \frac{48\times 9.75\times 1{,}200{,}000}{5\times 360\times 1200\times 2} = 130 \text{ in. or } 10.8 \text{ ft.}$$

$$Mr = 2\times 9.75^2\times 1200/6 = 38{,}000 \text{ in.-lb.}$$

* Deflection in this case exceeds 1/360 span.

20-in. wall, cast at the same time as the floor slab. The third was in a 6-in. partition wall, cast after the slab had been stripped.

The results of the first two tests seemed to indicate that the maximum pressure came when the head was about 3 ft. and diminished or remained constant thereafter. This was because the wall could not be poured faster than 3 ft. in height in 30 minutes, and the concrete rapidly lost some of its hydraulic effect after this time had elappsed. Up to the time the mercury column began to fall, the pressure was about 130 lb., but the head was so small the readings were unreliable.

The test on the partition wall gave good results, which averaged 125 lb. per sq. ft. This wall was filled in about 25 minutes. In this test also the gages showed a decrease in pressure after about 30 minutes.

It would seem therefore that 140 lb. per sq. ft. is a conservative value and, further, that the rate of filling the form should be considered in selecting the design head.

Timbers used as joists to support the floor forms are figured for the full live load of 75 lb. per sq. ft. plus the dead load of the slab, on jobs where side-dump cars carrying a four-bag batch of concrete are used for distribution. Table I has been prepared on this basis, using the simple beam formula. This will take care of the worst case and places the greatest factor of safety where it is most needed.

On jobs where buggies, barrows or chutes are used for placing concrete, the spacings and spans given in the table can be increased because the live load will be a great deal less. It is doubtful if the full live and dead loads act at the same time in any case.

TABLE II.—SPACING OF GIRTS IN FEET.

Slab Thickness, in. / Weight, lb. per sq. ft.[1]		3.0 112.5	4.0 125.0	5.0 137.5	6.0 150.0	7.0 162.5	8.0 175.0	9.0 187.5	10.0 200.0	11.0 212.5	12.0 225.0
Size of Girt, in.	[2]Span of Girt, ft.										
3 x 6	3	12.3	11.1	10.1	9.3	8.6	7.9	7.4	6.9	6.5	6.2
"	3.5	10.6	9.5	8.7	7.9	7.3	6.8	6.4	6.0	5.6	5.3
"	4	9.3	8.4	7.6	7.0	6.5	6.0	5.6	5.2	4.9	4.7
"	4.5	6.8	6.1	5.5	5.1	4.7	4.3	4.0	3.8	3.6	3.4
"	5	5.3	4.8	4.3	4.0	3.7	3.4	3.2	3.0	2.8	2.6
"	5.5	4.3	3.9	3.5	3.3	3.0	2.8	2.6	2.4	2.3	2.2
"	6	3.7	3.3	3.0	2.8	2.6	2.4	2.2	2.1	2.0	1.9
4 x 6	3	16.3	14.7	13.3	12.2	11.3	10.5	9.8	9.2	8.6	8.2
"	3.5	14.0	12.5	11.4	10.5	9.7	9.0	8.4	7.9	7.4	7.0
"	4	12.2	11.0	10.0	9.2	8.5	7.9	7.3	6.9	6.5	6.1
"	4.5	8.9	8.0	7.2	6.6	6.1	5.7	5.3	5.0	4.7	4.4
"	5	7.0	6.3	5.7	5.2	4.8	4.5	4.2	3.9	3.7	3.5
"	5.5	5.8	5.2	4.7	4.3	4.0	3.7	3.5	3.3	3.0	2.9
"	6	4.9	4.4	4.0	3.7	3.4	3.1	2.9	2.7	2.6	2.4

[1] Weight includes dead load of slab plus 75 lb. per sq. ft. live load.
[2] Span which will develop 1200 lb. per sq. in. fiber stress.
For 3 x 6-in. girts, $Mr = 35{,}000$ in.-lb. $= M_0 \times 1.2$.
For 4 x 6-in. girts, $Mr = 33{,}000$ in.-lb. $= M_0 \times 1.2$.

The span of the joists is usually fixed by the position of the girts. The location of the latter is determined by the column spacing in a mushroom floor and by the beam spacing in beam and girder construction. The spacing of the joists is kept as near 24 in. as possible in order to work the $\frac{7}{8}$-in. panel decking to the limit. With the span and the spacing known, a timber can be selected from the table as the joist size. The spacing of joists once determined should be kept standard for the job, so that when the panels are reused their cleats will not interfere with the joists previously placed.

Table II gives the spacings of posts under the girts for various spans of joists and thicknesses of slabs. These tables were calculated for a 24-in. spacing of joists. The concentrated loads from the joists were used on a

simple span in calculating the bending moment. The worst case, when one joist comes at mid-span, was used in this calculation.

It will be noticed that the full live load was also used in preparing this table. However, since the full live load will never get to the girt and the span of the girt is always continuous for at least two spans, the moment of resistance of the sticks has been multiplied by 1.2.

The load on the posts is next found. This load is usually far below its allowable load as a strut, but, as has been stated, the latter may cause an excessive crushing stress perpendicular to the grain in the girt at the top of the post. It is therefore well to play safe at this point, especially where granolithic finish is to be cast with the slab. A dish in the finished floor is often caused by settlement in the panel due to this cause. A drop of $\frac{3}{8}$ in. is not uncommon if soft wedges are used.

Deflection in the floor timbers themselves need cause little worry. As seen by the tables, fiber stress governs except in very unusual cases. When it is desirable to increase the spans given in the tables in order to make a better arrangement of centering with only a slight increase in fiber stress, it is well to compute the deflection.

Tables III and IV give the spacings of wooden column yokes for a bolt-and-wedge scheme. A fiber stress of 1800 lb. per sq. in. has been allowed and the load on the cleats determined by using the concrete at 140 lb. per sq. ft. water equivalent. The span of the yokes has been taken center to center of bolts, allowing $2\frac{1}{2}$ in. for column sides, the thickness of the yokes, and 1 in. for wedging. This gives a greater bending moment and heavier yoking than in some tables that have been proposed by others, but the construction of which the writer has not been able to check up.

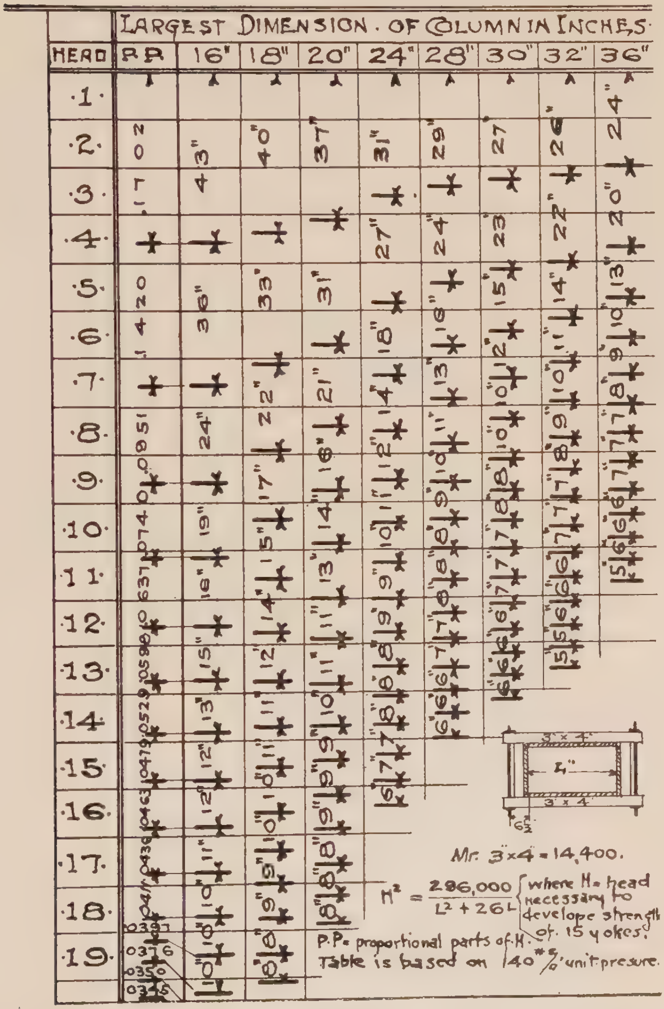
For example, one well-known author on concrete work in a published table gives five 2 x 4-in. yokes on edge for a 30-in. column with 10 ft. head. I have seen a column fail under similar conditions when yoked with seven 3 x 4 in. I conclude, therefore, that in making such tables the span of the yoke must have been considered as the concrete column size and a low concrete pressure used in finding the load on the yokes.

On our work we have used several styles of patented metal clamps and have always found that the manufacturer advocates a much greater spacing than we have found allowable on the job. In one case, we used eight adjustable steel clamps of a certain make on a 24 x 24-in. column 13 ft. 5 in. high. We noted considerable deflection in the clamps. We asked the makers for their recommendation for yoking a column of this size and height and their answer was "four or five clamps would be sufficient." We had practically the same experience with another make in regard to recommendation for spacing their clamps. I believe manufacturers would do well to make some study of actual conditions before putting their products on the market.

The deflection of the column sides between the yokes will never need attention except when very large yokes are used. The limiting span for boards is shown in Fig. 2. In most cases when yokes are properly spaced for strength and for action on the bolts the spacing will be well within the limits for the boards.

A cheap method of centering a wall 6 ft. high and over is to use full-height vertical panels, heavy horizontal studs, heavy vertical liners or standards and large rods threaded at both ends, with suitable plate washers to give proper

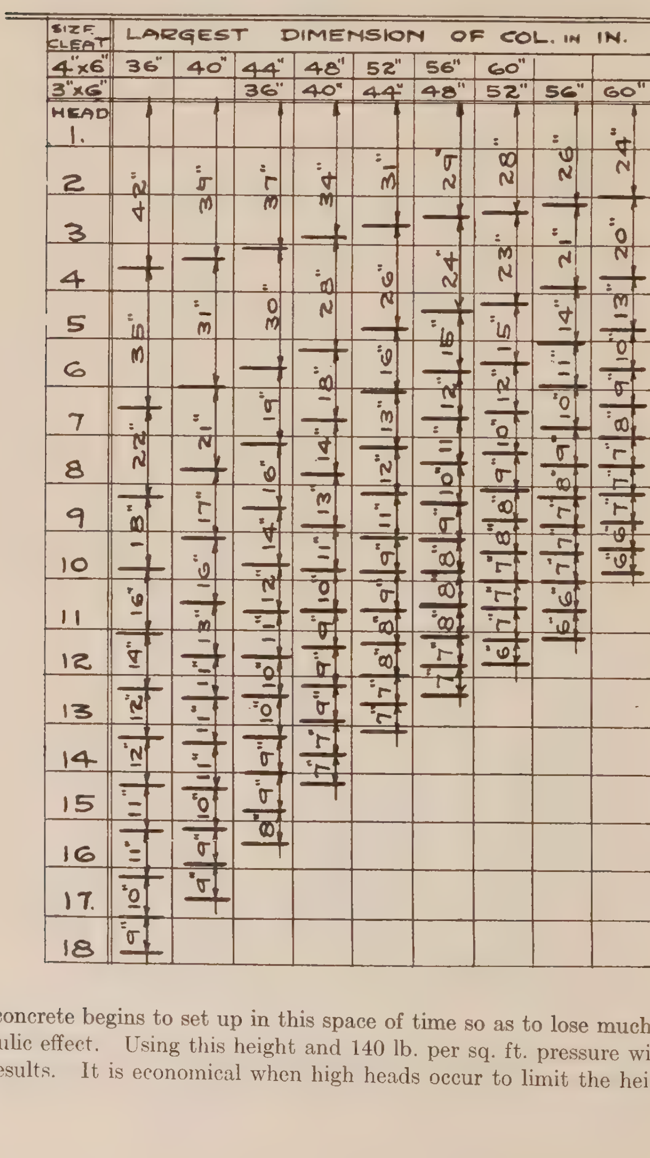
TABLE III.—SPACING OF 3 X 4-IN. WOODEN COLUMN YOKES FOR MAXIMUM FIBER STRESS OF 1800 LB. PER SQ. IN.



bearing for the load on the rod. By this scheme the stock can be stressed nearer to its allowable load and fewer ties used through the wall. Fig. 3 gives a typical cross-section for this scheme. There are so many unknowns

entering into calculations for wall centering that any tables are of doubtful value. It should be noted that as a rule the head of concrete in a wall is limited by the height that can be cast by the placing outfit in an hour. Ordi-

TABLE IV.—SPACING OF WOODEN COLUMN CLEATS FOR MAXIMUM FIBER STRESS OF 1800 LB. PER SQ. IN.



nary concrete begins to set up in this space of time so as to lose much of its hydraulic effect. Using this height and 140 lb. per sq. ft. pressure will give safe results. It is economical when high heads occur to limit the height to

which the concrete can be cast in order to use lighter forms. A system which requires a large number of rod or wire ties is decidedly uneconomical.

More important to the success of the job than the values used in the calculations are the methods employed in the drafting room in turning out drawings of forms accurately and speedily. The person in charge should be

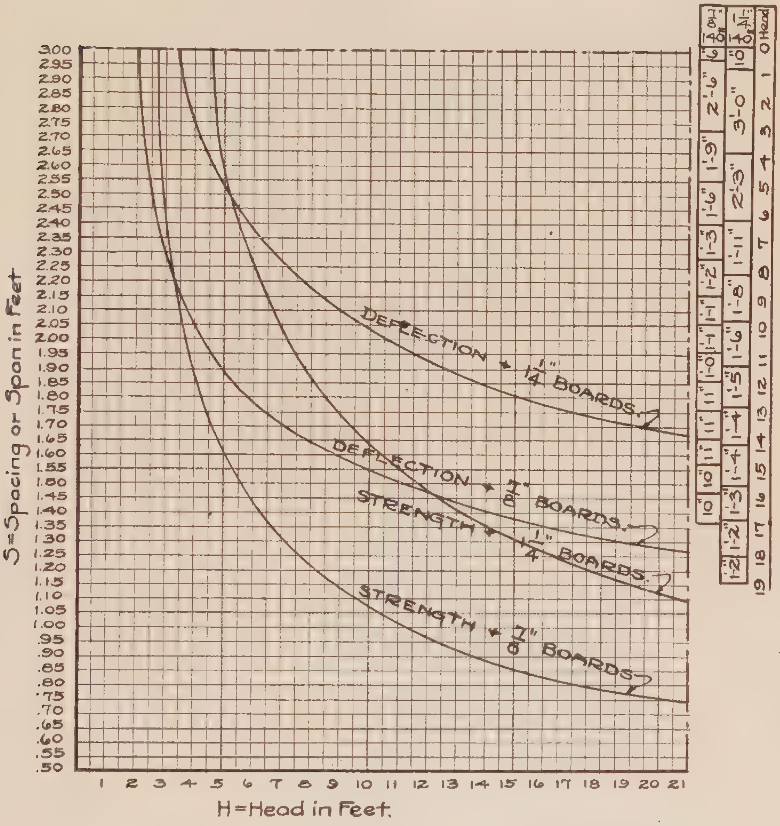


FIG. 2.—SPACING OF HORIZONTAL STUDS.

Maximum fiber stress, 1200 lb. per sq. in.; M=0.1 Wl; Deflection=3 $Wl3/384$ EI.

experienced in the different types of concrete building construction and should have men under him who are not simply "picture" draftsmen, but men whose judgment of building work is reliable. With such an organization form details can be prepared in advance in the office, leaving the job superintendent free to work out the many other details attendant upon the beginning of the job. The carpenter foreman can also use his whole energy in directing the carpenter

labor and the carpenters will need only to use their tools to assemble the panels made up in advance.

The first study and drawings to be made form a general assembly. Usually several different combinations of timbers and methods of assembling

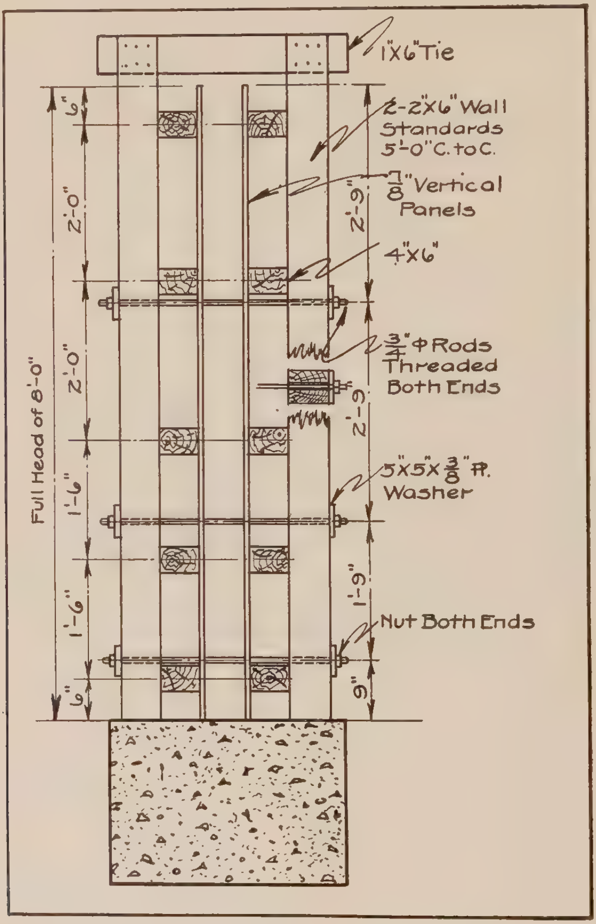


FIG. 3.—ASSEMBLY OF WALL FORMS.

the panels are sketched up and compared as to cost. Other things being equal, of course, the cheapest design is adopted.

A record of costs of the various units which go to make up the complete centering scheme is therefore necessary. Contractors who do not have such

a record will find Taylor and Thompson's "Concrete Costs" a valuable guide in this work.

Points to be remembered in this study are:

1. Joists and girts should be in as few lengths as possible to save time in sorting on the job.
2. Use stock sizes and lengths of lumber.
3. Keep number of panels and pieces to a minimum.
4. Provide easy stripping.
5. Allow clearance enough for slight inaccuracies in making up and erecting, swelling of panels, etc.
6. Panels should be a whole number of boards in width, if possible, for ease in making up.
7. Units to be as big as can be handled and joists used as panel cleats where possible.
8. Provide for reuse of panels.
9. Beams to be handled as trough units when the job is regular and units can be reused.
10. Consider use of floor domes or inverted boxes when beams are close together. In this way beam sides and slab are erected, stripped and moved as a unit. When either of the last two systems is used the beam sides should be given a slope to prevent hard stripping.
11. Provide for reshoring if necessary.
12. Have bracing above men's heads.
13. When four beam haunches occur at a column consider making haunches as a unit similar to a column head in flat-slab construction.
14. Consideration of steel forms.

The general assembly shows how the various parts are to be put together, supported, tied and braced to resist the concrete pressure and the weights coming upon the members. This drawing in its final shape can be made on tracing cloth with soft pencil. The dimension arrows, lettering and figures should be inked so that clear prints can be made to avoid errors in the field. These assemblies should be filed permanently for reference on future work. Fig. 4 shows a typical assembly for flat-slab construction.

When it is necessary to strip the floor centering for reuse before the concrete has been in place long enough to gain its full strength, provisions for proper support of the green slab must be made in the design of the floor forms. The safest and cheapest method of doing this, in the writer's opinion, is to place boards between the floor panels and wedge posts up to a bearing under these boards before the centering posts are knocked out. In this way the slab is never left unsupported as is the case when reposts are placed after all the centering is down. These boards should be placed according to a plan and located so as to shorten the spans of the main reinforcing bands.

After the general scheme for the forms has been decided upon, the detail panel drawings are prepared. By panel, in this paper, is meant several boards

cleated together into a unit to be used as a form for some part of a concrete member. Every different size panel is first sketched roughly on standard 6 x 9-in. sketching pads. These pads have holes punched at the top so that as sketches of the different kinds of panels are made they can be bunched together and brass rivets put through the holes. These sketches are transferred in more detail to thin tracing paper. The standard sheet is 25 x 33 in.,

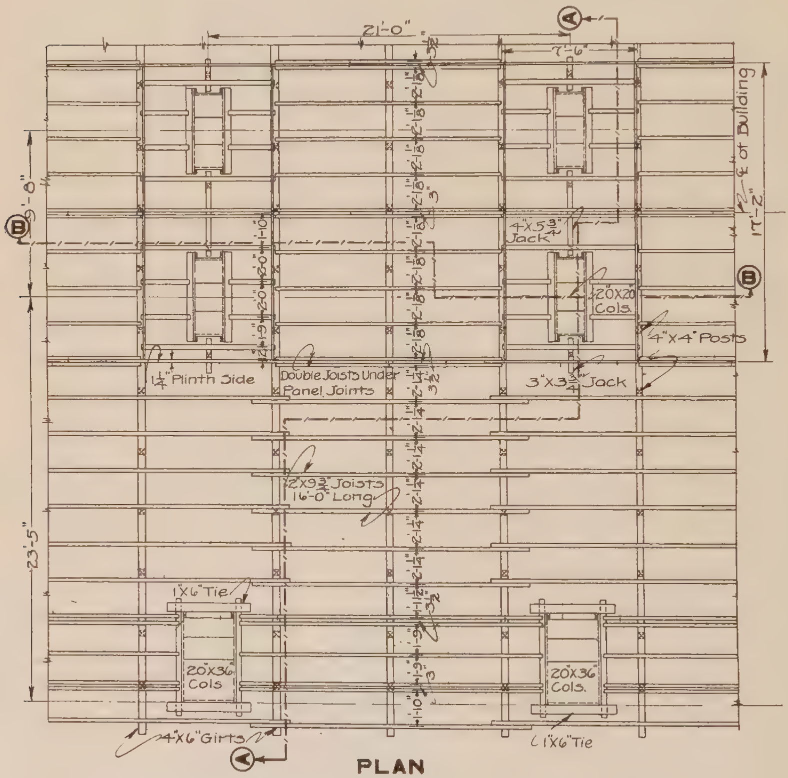


FIG. 4A.—TYPICAL ASSEMBLY OF FLOOR CENTERING.

divided by a 1-in. space into two halves of five spaces each. These details are used at the job mill for making up the panels. The blue-prints sent to the mill man are cut up into the 5 x 16-in. units, with one detail on each, and given to the carpenters at the "making-up" benches. On the detail sheet for each panel is given the panel mark, which is stenciled on the finished panel, with the number wanted and floor on which they are to be used. The sizes of the stock, dimensions, and alterations, when panels are to be reused, should be clearly marked.

A system of symbols as follows is easily learned by the workmen and should be kept standard:

B—Beam side.
BB—Beam bottoms.
F—Floor slab panels.
P—Plinth forms.
H—Haunch forms.
C—Column sides.
W—Wall panels.

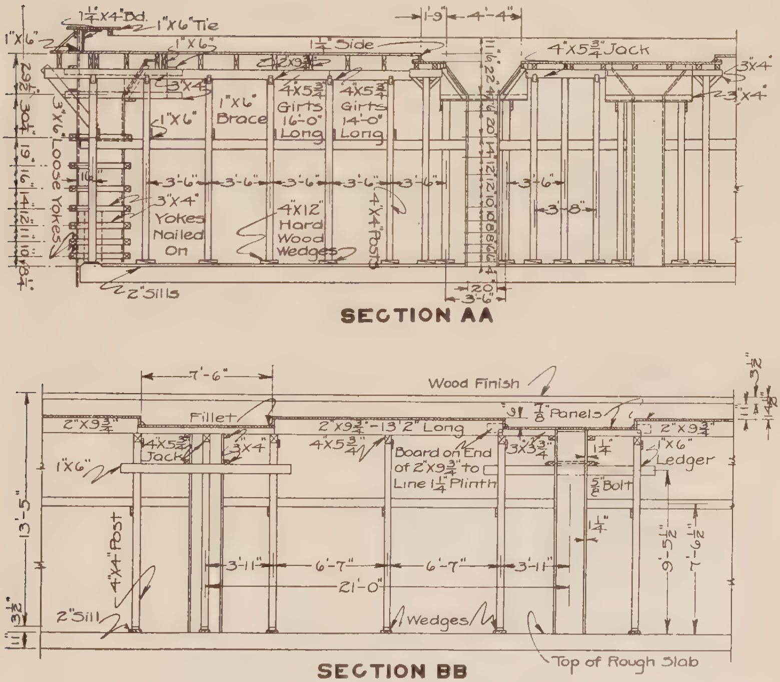


FIG. 4B.—TYPICAL ASSEMBLY OF FLOOR CENTERING.

Thus B18 means beam side No. 18 and its location is shown on the key plan.

These detail sheets can be drawn up entirely with soft pencil on paper as they are not valuable after the forms are once made. Rubber stamps for a great deal of the lettering will save time.

Where time allows and plans are complete, it is advantageous to get out one kind of panel at a time. In any case the first panels to be sent to the making-up mill should be the typical floor panels. These can be used eco-

nomically in building foundation walls, etc., and in this way the job can get an extra use out of them.

It is impossible to get a satisfactory cost unit for doing this form drawing. A price per square foot or per detail varies widely, according to the number of details required and the number of square feet that can be made from one detail in different buildings. A big building, where a large duplication of the different kinds of units is possible, will cost much less per square foot for this work than a small building which is badly cut up. The average cost per sheet has been found to be about $4.50 complete. This includes time spent on studies, assemblies, key plans, scheduling and checking. The number of sheets required on buildings of different sizes of the same type is fairly uniform according to the size. The number varies from 30 sheets on a small building up to 100 or more on a large building with irregularities. By reducing the cost of the sheets to drafting room hours by dividing the probable total cost by the average rate per hour, a fairly close estimate of the number of men required to turn out a job in a specified time can be made. This is useful on rush work.

The key plan mentioned above is really a diagram of the floor plan upon which is shown the location of the various form panels. This plan is the only one which the workmen consult in erecting the form work. It must, therefore, be complete and clear.

This plan can best be made by tracing the floor plan on cloth from a blue-print, indicating the beams and holes in the floor and the columns and walls in the story below. The lines should be heavily inked and the figure large enough and spaced so that the tracing can be photo-reduced to convenient size for the foremen to handle on the job. A big blue-print is inconvenient and fades in the strong sunlight. As a rule, to avoid misunderstandings, a key plan should be prepared for each floor in the building.

The key plan may be supplemented by a letter to the job in which should be noted assumptions made in preparing the details. These notes might include such information as clearances allowed, grades at which forms are to start, scheme for reuse, etc.

Fig. 5 shows a convenient form to use in scheduling the panels. A schedule is made for each different kind of panels. This form is especially useful on a rush job where forms for one floor at a time must be completed. The additional number of each kind of panels already detailed, to be made up for each floor after the first, can be shown on this schedule.

Materials.—The materials to be used for the form work should be scheduled from the form drawings and not ordered by guess, as is so generally the custom. If complete form drawings for the job are available in time, it is economical to order even the boards for column and beam forms to exact widths and lengths to make up the panels. This saves ripping and cutting off the boards in the job mill, where the saws are often taxed beyond their capacity at the beginning of the job getting out miscellaneous stock.

It is always desirable to get the lumber in car-load lots because of the lower price. For this reason, on many jobs, in order to get the stock in time

from mill shipment, it is necessary to prepare the schedule from the general assembly and typical details.

The joists, girts, posts, braces and sills can be taken from the general assembly plan directly by multiplying the stock required for a typical bay by the number of bays in the building and the number of floors of centering needed. Girts and joists should be ordered edged in order to obtain an even bearing for these members.

The board feet of $\frac{7}{8}$-in. roofers for slab and wall panels can be taken off directly from the square feet of floor and wall surface, adding at least 10 per cent for waste and 25 per cent for cleats. As these come in random lengths it is desirable to specify the length preferred, so as to get as big a percentage as possible of this length.

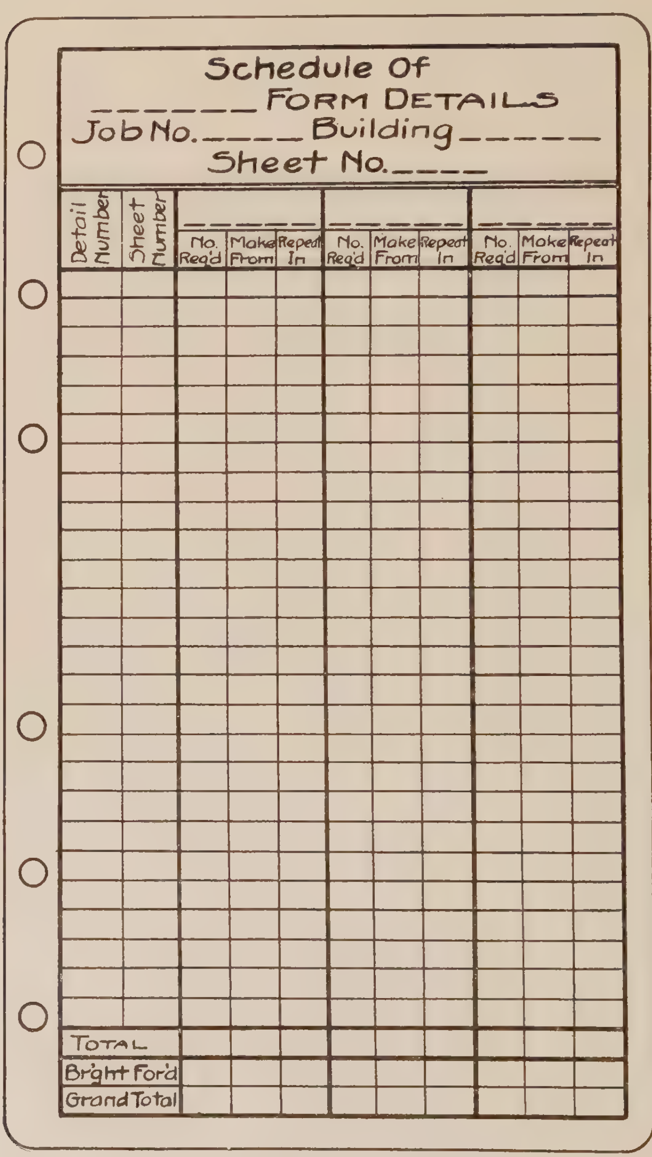
One and one-half inch boards for column and beam sides and 2-in. plank for beam bottoms should be ordered P-4-S. Boards $7\frac{3}{4}$-in. and $5\frac{3}{4}$-in. wide after planing are the most useful widths for beam and column sides and can be ordered in random lengths, specifying again the lengths desired for as big a percentage as possible. The best length for the majority of columns and beam sides and the best width for beam bottoms can be easily taken from the plans by one familiar with form work.

Stock for beam cleats and column yokes can be ordered rough and the quantity can be closely estimated from the assembly plan. Rough plank and timbers for staging and runways should also be included in the order.

By getting a schedule of the form lumber in this way as soon as possible after the contract is secured, there is usually time for a thorough canvass for prices and delivery. At the same time the contractor knows that the quantity called for on the lumber schedule takes care of and is suitable for the needs of the job. In this way the very lowest price is secured on the majority of the stock needed for the building.

From the assembly of the columns and walls a schedule of the number and lengths of rods and fittings needed for tying the forms together can be made and the order placed early. It is safer and more economical to use rods threaded at both ends. The thread at one end can be short for a hand nut while the other end can be threaded to give any desired adjustment. A nut and washer can be set by a laborer at the proper point on the long thread so that the carpenters need only use the hand nut on the short thread. If a long thread on each end is desired, a nut can be turned up and loosened by a special socket wrench operated like a carpenter's bit stock.

The use of check nuts on a rod to give adjustment in length, depending upon a set screw for its grip, while convenient in some ways is not economical in the long run. The grip of the check nut is the weak point of the outfit. This makes it impossible to stress the rods to their working strength. If, however, a set screw is overlooked and not tightened at all a failure is liable to result. The worst that can happen when threaded rods are used is a crushing of the wood under the washer if the bolt is over-stressed. The washers used, of course, should not be the standard cast iron variety but should be steel plates of sufficient area to develop the strength of the rod when the washer is bearing on wood.


Schedule Of
_______ FORM DETAILS
Job No.____ Building______
Sheet No.____

Detail Number	Sheet Number	______			______			______		
		No. Req'd	Make From	Repeat In	No. Req'd	Make From	Repeat In	No. Req'd	Make From	Repeat In
TOTAL										
Br'ght For'd										
Grand Total										

FIG. 5.—BLANK FORM FOR SCHEDULE.

The possibility of using metal forms must be considered when planning the form work for any concrete structure. Where it is possible to use the same units many times without remodeling, or by making adjustments easily provided for in the metal forms, the latter are usually an economy. On sewers, pipe lines, subways, straight wall work, etc., they can usually be used to advantage. In building work the use of metal forms, except for round columns and column heads in flat-slab construction, is of doubtful economy.

It is usually impossible to get enough uses out of the metal forms on one building to make the first cost, even on lease, compared favorably with wood. Then, again, the building must be designed for the standard form units or else there is the expense of buying special units which are of no use after the building is done.

The company with which the writer is connected has recently used one well-known make of sheet metal forms on the exterior walls of a group of five buildings having a total of 267,000 sq. ft. of contact surface. These wall surfaces were at first thought ideally adapted to metal forms, but when the elevations were laid out and the task of fitting standard units to them begun many obstacles were encountered on account of irregularities. Many special panels had to be purchased and a great deal of wood used for the fitting. By this office study many problems were encountered and solutions found long in advance of erection, thus saving time and expense on the job.

On this job the forms were used an average of $13\frac{1}{2}$ times at a labor cost of 9.23 cents per sq. ft. for making, erecting and stripping. The total cost, including material and plant, was 12.93 cents per sq. ft. These figures include office expense of the job and were made by an organization enthusiastic over the use of steel forms and are as good as may be expected on general work.

One bad feature we found with the particular form we used was the cost of purchasing extra wedges and keys. Of a total cost of 2.7 cents for material, 0.5 cent was for extra keys. These were loose parts easily lost. There are makes of steel forms which do not have this disadvantage and for this reason might reduce the cost.

The steel forms on these buildings produced a good exterior, true to line, and required less rubbing down after the forms were stripped than is usually the case with wooden forms.

Sheet metal can be used economically in forming an octagonal head at the top of a square column. When the column sizes do not change in different stories the design is simple. We have, however, used successfully one set of metal heads on square columns ranging from 38 in. in the first story to 20 in. in the top story. The adjustment was made by using a plate on each side of the head in which were punched lines of bolt holes spaced for the reductions. Black sheet metal, No. 22 gage, is usually sufficient for these heads and can be easily worked by the sheet-metal men.

There are several schemes of patent adjustable metal centering and column yokes on the market which have merit for particular cases and should be investigated when considering form material. Material of this kind, which can be rented, is more attractive to the contractor who dislikes to fill up

his plant yard with equipment not always usable and to pay freight charges when lumber can be picked up locally, quickly and cheaply.

Corrugated sheet metal for slab forms can be used economically on some buildings, especially those of the flat-slab type. The corrugated ceiling, however, is objectionable to some owners. When a large number of inserts need to be placed the use of metal sheets is of doubtful economy because it is more difficult to attach the inserts to the metal forms.

The sheets are laid over the wooden joists, lapping the metal at the ends and sides. These sheets can be reused many times and recorrugated on the job. When unfit for forms the sheets can still be used for concrete chutes and as siding for temporary buildings on the job.

Shop Work.—The saw mill should be located, after a study of the job site, in a position where there is plenty of space for piling the stock and also the finished panels. The saw-mill shed and equipment should be standard so that it can be erected and put in operation as soon as possible in order to reduce hand-sawing to the minimum. A saw mill at its best is dangerous. For this reason every precaution should be taken to protect the workmen by efficient saw, machine and belt guards. The mill should be near the building, so as to reduce the cost of moving panels. This moving cost is an important one in obtaining low erection costs and will amount to a considerable item when the mill is poorly located, either from lack of planning or from lack of space for the plant around the building.

The mill yard should be so arranged that the stock will go in one direction from the lumber piles to the saws, the making-up benches and to the finished panel piles, which should be nearest the building.

The lumber when received is checked as to quality and quantity. The stock is then sorted and piled according to size and length, each pile having its size plainly marked.

It is then an easy matter for the mill man to make a lumber ledger of all the stock in his yard. Thus at all times it is possible to know the stock available, for as the lumber is used deductions can be made and the running total kept up to date.

The mill man divides the panel details up into boards to make the required width of the panel. The number of boards of each kind needed to make the number of panels wanted are ordered moved to the saw to be cut to proper length, and thence to the make-up bench, where cleats of the proper size and length are already waiting, having been previously ordered through the mill. Another order to the bench carpenters, which is clipped to a blue-print sketch of the panel, enables them to cleat together the boards into the finished panel. The panel is then taken away by a laborer, stenciled with its location mark, oiled and piled until ready for use in the building.

All this is done by orders written on standard order forms of three kinds, one for moving the stock, another for sawing and the last for making up. Duplicates of the orders are placed on the mill man's progress board so that he knows at all times how the work is progressing, for when an order is completed and returned to him its duplicate is taken down. Each kind of order is of different color so as to be easily identified.

There are several points in connection with making up panels which may be considered here.

As much assembling as possible should be done at the bench. Rangers for wall beams can be attached and ledgers to support joists can be nailed to the interior beam side cleats at the proper depth. When inserts need to be placed in the sides of beams, the holes for the bolts to hold them can be located on the details and the holes bored at the mill. The groove strip for steel sash and also the corner fillet can be put on the column sides and beam bottoms. Bevel kep strips for walls should also be nailed lightly to the panels when necessary.

Heavy cleats for big wall columns should be cut and bored, but not attached to the panels. The panel can be cleated with $1\frac{1}{4}$-in. boards, and is thus made much lighter and easier to handle in erecting. Clean-outs at the bottom on two opposite sides of all columns should be made at the mill. Reduction strips should be nailed at the edge of all panels which reduce in size when reused.

All small pieces liable to get lost or used for other purposes can be dipped in red paint so that their small size will be respected by the carpenters when looking for loose boards.

Much waste in making up $\frac{7}{8}$-in. panels can be prevented if the floor is laid out so as to make the majority of the panels a whole number of boards wide. Roofers come $5\frac{3}{8}$ in. to $5\frac{1}{2}$ in. wide, and it is an easy matter to plan the panels to come, say, seven or eight boards wide, which means no waste in ripping one board to make the width. The length should be planned as near stock lengths as possible. Panels made of spliced boards are expensive to make and easily broken.

It is not economical in labor to make up panels from lumber which has already been in contact with concrete several times. On a large building recently erected by us the average cost of making all panels from new stock was $1.10 per square. On another building of exactly the same type the average cost of making all panels was $1.43 per square. On the latter building old lumber from the first was reused in making a large part of the panels. These prices include oiling, lumping, saw filing, carpentry, sawing and job mill overhead.

If, however, panels are in good condition they can be cleaned and repaired at a considerable saving, both in labor and material. As an example, 13,562 sq. ft. of $\frac{7}{8}$-in. floor panels were cleaned, repaired, cut to a new length, oiled and piled; 6560 sq. ft. of $1\frac{1}{4}$-in. column sides were cleaned, repaired, oiled and piled; 1250 sq. ft. of column heads were cleaned, repaired, oiled and piled at a total cost of $99.25, or a unit labor cost of $0.00466. The estimated labor cost of making the above forms from new stock, using unit costs obtained on the same work, was $178.20, or a saving of over 40 per cent in labor, besides a big saving in material.

Old $1\frac{1}{4}$-in. panels can be knocked to pieces, the boards cleaned and piled for about $3.00 per 1000 ft. B. M. Large quantities of typical forms can be made from these boards at a slight increase in labor cost over using new stock. The lumber saving is considerable. The quantity made must be large to allow a thorough routing of the old stock through the mill.

The cleat spacing for the various kind of panels should be kept uniform so that the strips on the benches, for spacing the cleats, will not need to be moved for every set of panels.

For beam sides 2 x 3 in. cleats, flat, can be used on panels up to 30 in. deep; 2 x 4 in., flat, up to 42 in.; and 3 x 4 in., on edge, above 42 in. deep. Nails should be specified about as follows:

No.	Size.	Width of Board.
3	10d	$6\frac{3}{4}$ and $7\frac{3}{4}$ in.
2	10d	$2\frac{3}{4}$ to $5\frac{3}{4}$ in.
1	10d	Less than $2\frac{3}{4}$ in.
. .	12d	In 3 x 4 in. cleats.
. .	8d coated and clinch ends	In $\frac{7}{8}$ in. boards.

Wire nails should be used for form work because of ease in driving and drawing. The holding power is also sufficient. Double-headed nails should be used in securing all boards which have to be loosened before stripping.

Field Work.—All work on the forms after the finished panels are piled at the mill will be called field work to distinguish it from the work in making up the forms, which has been called shop work. Under this head will come everything connected with using the panels on the building. An important feature of this work is a planning department, whose business it is to plan the work in advance so that at all times the several gangs will have definite tasks to do and be supplied with sufficient material with which to do the work.

An editorial in *The Contractor* of May 15, 1915, says in this connection: "This planning consists of many things. First, it demands complete plans and drawings for the work in hand. Plans cannot be made for building a structure unless the details of design have been prepared and from these the kinds and quantities of work are known." In the writer's own work, complete plans are rarely available when work begins and usually only very preliminary drawings have been prepared. Nevertheless the work should be planned, using experience on past work and available information on the job at hand as basis to work upon.

The moving boss has an important position in this field work. He is responsible for getting the proper panels to their correct location in the building and having all other stock called for on the assembly plan on the job ahead of the erection carpenters. When the forms are stripped, he must move the panels, which are to be remade before their next use, to the remaking benches and thence to their next location. It is expensive to have high-priced carpenters waiting for stock or using stuff not suited for the job they have to do.

The erection of the centering and the assembling of the panels, after the latter have been moved to their proper location in the building, is done by carpenters. It is economical to employ good carpenters and keep them employed constantly. A gang of men that understands form work on concrete buildings, the methods used and the grade of work desired, is a valuable asset for any firm specializing in reinforced concrete work.

A competent foreman should be in charge of all carpenter labor. He should be consulted by the planning department so that "team work" will prevail.

Quality in concrete building work is usually more desirable than low costs. Good lines and surfaces will be remembered after the cost is forgotten. These results can be obtained only by careful supervision of the erection of forms.

When forms are to be reused, as is usually the case, stripping should be done as carefully as possible. An intelligent foreman in charge of a stripping gang is a good investment. Any man can wreck forms cheaply, but a man who can strip the forms and leave them in good condition for reuse is, in the end, the cheapest stripper. He saves the time of the high-priced carpenters in remaking and fitting broken panels.

The question of when to strip is one for the building designer to decide, and his instructions should be carefully followed. It is cheap insurance to make enough forms so that no chances need be taken of weakening the structure by stripping while the concrete is still green.

The panels which need to be altered before reuse should be remade from the blue-prints showing the necessary changes. A bench for this purpose can be set up in the story where the forms are stripped. It is sometimes advisable in a high building to set up power saws in one of the upper stories for cutting off panels and getting out stock for remaking and repairing.

A large gang of carpenters working overtime is sometimes necessary in order to deliver a building on time. This is expensive, however. The workmen cannot give back in efficient labor the value of their high wages. Ten hours is about all the average man can work without falling off greatly in efficiency. Night work on forms should be avoided if possible.

When speed is not the essence of the contract, there is time to plan the erection work more carefully. Smaller gangs can be used and their tasks and material for them routed by slips similar to those used in making up panels.

An important item in the field work is the erection of economical safe stages. These stages should be carefully designed as to the worst possible load to come upon them, and the design should be strictly followed in the field. A man thoroughly familiar with lumber should be made inspector as to the quality of the stock used for stages. A good stage makes the workmen more efficient and is a good insurance investment.

General Notes on Form Work.—The keynotes to economy in a concrete building are regularity and simplicity. These two points, if borne in mind by the designer, make cheap forms. The latter should have the idea of forms as firmly in his mind as he does his bending moment formulas, if he is to turn out a good reinforced concrete design. Concrete used with brick curtain walls can be worked into very pleasing exteriors without the need of elaborate details requiring expensive forms.

Projecting belt courses break up the continuity of the exterior column forms and should be avoided except on very high buildings. A heavy overhanging cornice, consoles and brackets needed at the top of a tall building can be more cheaply made of sheet metal and attached to a plain concrete cornice beam by means of steel brackets. The effect is just as pleasing as if expensive forms are made and the concrete members cast.

The number of different sizes of beams, columns and thickness of slabs

should be kept a minimum. This means fewer different kinds of units. The panels can be made and handled more cheaply, to say nothing of the time saved in designing. The biggest item of expense, however, is that of remaking beam and column forms and patching floor forms when changes occur in the size of the members on different floors. It may be nice to know, for instance, that an 18 x 18-in. column is sufficient to carry the load, but is it better to know that a 20 x 20-in. column, as used below, is the size to use ordinarily. This saves remaking the column sides and patching the beam and slab forms. In most cases concrete is cheaper than altering forms.

Stock sizes of lumber should be kept in mind when dimensioning the concrete members. For example, projections should be made $1\frac{1}{4}$ and $1\frac{3}{4}$ in. instead of 1 and 2 in., since dressed stock comes nearer the former sizes. Beams can be made $11\frac{3}{4}$ in. wide instead of 12 in. Thus one 12-in. plank can be used without needing a $\frac{1}{4}$-in. strip tacked to the edge. Points such as these, thought out in the designing room, mean a cheaper building.

It is economical on jobs where speed is not required to place concrete with light equipment and slowly. Lighter forms can then be used because the live load is greatly reduced when tracks and big cars do not need to be supported by the forms.

Conclusions.—Much has been written, many experiments made and standards adopted in relation to concrete and reinforced concrete design. It is the writer's opinion that more accidents can be prevented and money saved by contractors and for owners by showing the same interest in the study of form work. The greatest proportion of the small number of failures of concrete structures is due either to faulty form design or because the forms were stripped too early.

Recently we have had reported in the engineering periodicals the failure of a concrete slab, resulting in the death of two workmen. According to an expert's report "the form supports were faulty." Forty-five square feet of floor were supported on each post, made up of one long 2 x 4 in., to which were spiked short pieces of 2 x 4 in. to make up a 4 x 4-in. post. These posts had little bracing. Two floors of green concrete were supported on such posts. Is there any wonder that failure resulted?

As has already been stated, the unit cost of making a thorough study of forms for a job is difficult to determine, but it is small compared to the time saved in the field by using the more efficient methods. In spite of the fact that carpenter's wages have steadily increased, while the quality of labor has decreased, and a poorer grade of lumber costs more than a few years ago, unit costs on form work are much lower than when the job was left to itself. The overhead cost for this office work is a very small part of the total unit cost of the form work.

It is safe to say that by using methods such as have been described, a large saving is possible over doing the work by ordinary methods. If this paper helps to create further study and discussion upon the subject of concrete form work, in your Society, it will, I believe, have justified its existence.

DESIGN OF REINFORCED CONCRETE FOOTINGS FOR BUILDINGS.

By R. L. Bertin.*

The use of concrete, plain and reinforced, for the construction of foundation for all types of structures, has replaced almost exclusively all other materials, such as wood, brick or stone masonry and structural steel.

In the first application of concrete to foundation, it was used as a leveling bed, on top of which brick or stone masonry footings, cast iron bases or structural steel grillages were built or set; the function of the concrete being simply to transfer the direct vertical stresses from the bases of whatever nature to the sustaining soil. About 1880 it became a general practice to replace the masonry footings by concrete. These footings were designed on the same basis as the brick or stone masonry ones, the thickness being made equal to twice the projection beyond the face of the bases resting on them. However, owing to the low compressive stresses permitted on concrete by the various building regulations, these concrete bases were generally capped with granite or cast iron bases in the case of independent footings. As the quality of cement improved, and more faith was placed in the value of concrete as a building material, the idea of embedding steel in concrete was introduced and developed by the pioneers of reinforced concrete. After years of experimenting, the action of steel in concrete became better understood, and certain formulæ were evolved governing the design of reinforced concrete members, including footings. In view of the saving effected through the use of these footings as compared with the former methods used, reinforced concrete footings gained favor among architects, engineers and builders.

Owing to the lack of experimental data obtainable which could be applied to the condition of footings, wide differences existed in the methods of designing used by engineers engaged in the designing of reinforced concrete. It was not until 1913 when the results of reinforced concrete wall and column footing tests, conducted by Prof. Arthur N. Talbot, of the University of Illinois, were published in the University of Illinois *Bulletin* No. 67, that any light was thrown on the behavior of reinforced concrete footings under stress.

The factors established as a basis for the design of footings by Prof. Talbot as a result of the careful analysis of the data obtained from the tests of 147 specimens, are as follows:

1. The position of the critical section of bending moment.
2. The proper location of the center of gravity of the upward load causing bending about the critical section.
3. The proper distribution of bars in the width of the footing cutting across the critical section of bending.

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4. The position of the critical section for diagonal tension.
5. The distribution of bond stresses along the reinforcing bars.
6. The effect of using bars of different lengths for reinforcement.
7. The effect of bending or hooking the bars both vertically and horizontally.
8. The effect due to the use of stirrups on the resistance of footings to diagonal tension.
9. The effect of sloping the top surface of footings from the loaded area to the edges.

The section coinciding with the face of the pier or wall resting on the footing was found to fairly represent the critical section for bending moment.

The portion of the upward load causing bending about the critical bending moment section for columns is proportional to the trapezoid having for its bases the width of the pier for one and the outer edge of the footing for the other, the height being the projection of the footing beyond the pier face.

In order to find the center of gravity of the above-described portion of the total upward load, the trapezoid is divided into a rectangle, having for width, the width of the column face, and for length, the projection of the footing, and two triangles the altitude of which equals the length of the rectangle, and for base, one-half the difference between the total width of the footings and the width of the pier. The center of gravity of the load represented by the area of the rectangle is one-half the length of the rectangle, and the center of gravity of the load represented by the area of the triangle was found to be located at a distance equal to 0.6 of the altitude of the triangle. While this may seem ambiguous, one must remember that the triangle represents the amount of upward load which reacts on the part of the footing under consideration, but does not represent the distribution of this load along the footing projection.

Using the following nomenclature: (see Fig. 1.)

w = Unit upward pressure on footing base.
l = Side of footing.
a = Side of column.
c = Projection of footing beyond the face of pier.

The formula for the bending moment at the critical section may be expressed as follows:

$$B.M. = (1/2ac^2 - .6\ c^3)w.$$

And made equal to the usual formula for resisting moment

$$R.M. = Afjd.$$

Where A is the reinforcement in the given direction for a predetermined width of beam; f the maximum unit tensile stress; d the depth from top of footing to center of bars; j the ratio between the resisting moment arm or distance from center of steel to center of gravity of compressive resistance and d.

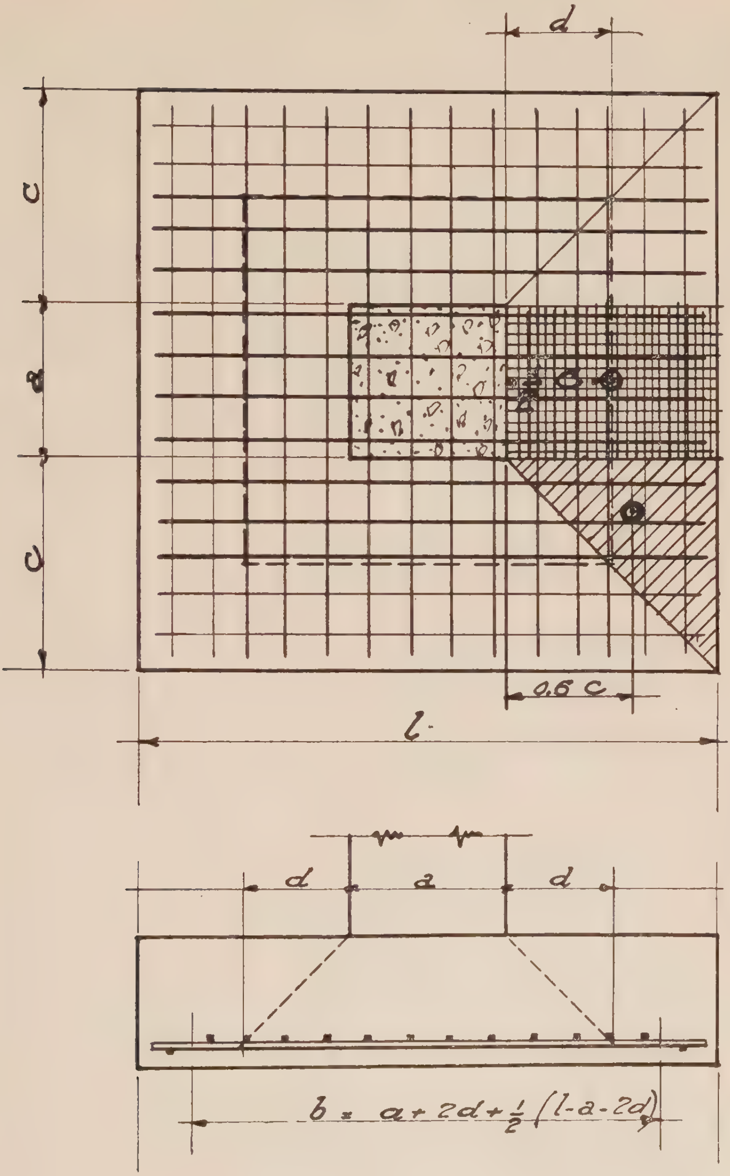

FIG. 1.

The distribution of bars in the width of the footing proved to be of importance, the conclusion derived from the tests was that the total reinforcement obtained from the above formula should be grouped in a width of beam b equal to the width of the pier plus twice the depth of the footing to the reinforcement plus one-half the remainder of the width of the footing l, or

$$b = a + 2d + 1/2\ (l - a - 2d)$$

The limiting value of b being l.

One of the most important factors established by the tests was the location of the critical section for diagonal tension; almost all the failures in diagonal tension showed cracks starting at a distance from the face of the pier equal to the depth of the footing from the top surface to the center of the steel and inclined at an angle of 45 deg. towards the pier face. The diagonal tension being measured in terms of the vertical shear V at a distance d from the face of the pier and considered to be that part of the upward load on the footing outside of the section considered, and is expressed as follows, for square footings:

$$V = (l^2\ (a + 2d)^2\ )w$$

and the critical unit vertical shearing stress v

$$v = \frac{V}{4(a + 2d)jd.}$$

The values of v obtained from the result of the tests which failed by diagonal tension agreed fairly close with those obtained in beam tests when checked in accordance with the above formulæ.

In determining the unit bond stresses, the upward load used in determining the bending moment is used; in other words, the vertical shear along the face of the pier. The usual bond stress formula being applied:

$$u = \frac{V}{mojd}$$

where u = unit bond stress;

m = the number of reinforcing bars used in resisting the bending moment at the critical section;

o = the periphery of the bar.

It has been the practice of many designers to compute the bond stress using one-half length of the bar as a measure of resistance, assuming that the unit bond resistance was constant along that length. This practice was not substantiated by the tests.

The practice used by some designers of staggering short bars for reinforcement instead of using full-length bars, particularly in footings having short projections in proportion to their depth, was shown to greatly increase the bond stresses and cause premature bond failures.

The use of stirrups and bent bars in the footings increased the resistance to diagonal tension, while the practice of stepping or sloping the top surface

of footings showed a marked increase in the intensity of diagonal tension stress.

The above observations are being adopted by a great many engineers as their basis of footing design, and the general adoption of the above premises for standard practice should be encouraged in order to eliminate the wide differences which heretofore have existed in the designs of reinforced concrete footings and which, owing to the more or less sound assumptions made, may be the cause of undue settlement of the superstructures and possibly their failure.

I would therefore recommend that, in connection with the work of standardization of reinforced concrete designs, footing designs be included and rules formulated in accordance with the above described observations and deductions.

MOTION PICTURE STUDIES ON THE MAKING AND PLACING OF CONCRETE.*

By Nathan C. Johnson.

The value of motion pictures in the study of making and placing of concrete is very great. A motion picture of an individual operation shows that operation *as it is*, not as it is supposed to be, makes plain the various distinguishing and important features incident to it, and preserves them in instant detail for analytical study. For example, the duration and sequence of mixing operations was thus shown by means of a time-clock to the sixteenth part of a second; and by making a large number of these studies, it was found that the average operations of charging and discharging the mixer are so slow that on the ordinary commercial schedule of batch deliveries the average mixing time does not exceed 17 seconds, and that in many cases there is practically no mixing interval at all. In this latter instance, the raw materials are fed into the mixer drum and discharged at the other side as rapidly as they can pass through the machine. The uncertain quality of concrete produced in these ways is beyond question, and the motion picture camera furnishes proof positive as to the existence of these conditions.

In like manner, the cycle of operations inside the mixer of the ordinary type is shown in its several phases. Beyond question, it is important to have concrete as dense as possible; and to this end there should be as little air as practicable churned into the mass during the mixing operation. The motion pictures showed, to the contrary, that in the usual type of mixer the inherent actions are of such a nature as to make this churning a maximum with a corresponding impounding of air and like detriment to the concrete.

In developing the argument of this lecture slides were thrown on the screen, showing first typical examples of defective concretes, particularly concretes subjected to water action. The nature of the problem of obtaining enduring concretes was explained in detail—the leaching out of the several substances composing the bond between the sand and stone and with consequent loss of strength and deterioration, and the causes resulting in the production of concrete so inferior in average strength as to make the use of expensive aggregates an economic folly. Photomicrographs of concrete sections were used to show the internal arrangement of the mass, with the dispersion of aggregates by excess water and the isolation of cement particles through their dispersion by the same medium. The expansive power of properly hydrated cement was also shown, with its void-filling capacities, if only it could be fully utilized; and the necessity was pointed out for a better type of mixer which would, by its own actions, have at a minimum the undesirable qualities of the present type of machine and the desirable qualities outlined at a maximum.

In the motion pictures, the prevalence of excess water was shown in a

*Note.—Owing to the impossibility of having in text more than a comment on the motion pictures as presented, this paper is printed in summary only—Ed.

variety of studies. In concrete road work three different pavers were shown in action with the excess water made most evident by the run-off as the concrete was deposited, leaving the stone almost bare at the top of the slab and putting the greater quantity of cement at the bottom of the slab—precisely in the reverse order from that which is most to be desired. The long charging time required and the short mixing interval allowed in commercial work were also made evident in these studies, and they carry a most wholesome lesson to those who are disposed to heed it. Excess water and short mixing time are quite universally condemned, but it is regretably true that those who are most free to condemn such practices feel equally free to follow them in daily work.

The physical characteristics of water and its relation to the other materials of concrete was shown by means of motion pictures of mixing operations in glass tanks. The inevitable attachment of air to aggregate and the partial isolation of the aggregate from the cement by such air-bells was shown by the behavior of sand grains in the water. So strong was this attachment of air to many of the sand grains that they seemed to rise through the water as though carried by small balloons; and the influence of these several factors on the character of concrete was pointed out, with particular emphasis being laid upon the influence of water in concrete, both as to quantity and quality. It is probable that no one factor is so responsible for poor concrete as the use of excess water, and by the use of these motion pictures this fact was brought home very clearly.

Not less important than the mixing operations are the processes of depositing the mixed materials in forms. A large number of studies of placing concrete were made, showing both the use of concrete barrows and of spouting. Views inside the forms were also shown, illustrating depositing from barrows, this deposition necessarily occurring in isolated places. Isolated delivery of this character results in a heaping-up of the materials at localities immediately below the barrow runways, the dependence for distribution of materials being placed very largely upon hoeing-down by men within the forms.

In work of this character, excess water again plays a very important part. Not only must dispersion of cement particles and sand particles and larger aggregate particles be brought about through the presence of excess water in the commonly-used sloppy mix; but furthermore, by reason of this heaping-up of materials at one locality, the more fluid portions of the concrete necessarily gravitate to the lowest part of the forms. These fluid portions in running off carry with them the greater part of the cement and the finer sand, leaving the stone almost bare; and in so great quantities were these fluid portions, that the man engaged in hoeing down waded almost to his knees in wholly fluid material and was able to put his arm into the fluid well above the elbow before touching any solid materials. Necessarily also, this fluid material carries on its surface a large quantity of laitance or scum; and at the end of a day's work, this laitance forms a dividing plane of little or no adhesive value, on the top of which is placed the succeeding day's deposit. These pictures made very evident the causes of "day's works planes" which are so evident in all concrete structures, especially those submitted to the leaching action of water, and are

a decided factor in the production of inferior concretes. Necessarily, also, there must be segregation of the heavier materials, with resultant inferiority in some portions of the concrete and irregularity and uncertain quality throughout the mass.

Studies of spouted concretes were exceedingly interesting. Again the tendency to use excess water in order to secure an easy-flowing and lubricated mass was made very evident. Studies were made of the concrete in the chutes, showing segregation due to this cause, with a great rush of material in the first part of the batch, with later a slow dribbling of the heavier portions throughout the length of the chute and their separation from the finer portions of the concrete. This again cannot help but result in uneven texture and uncertain quality.

By means of elevating the spout, the effect of spouting concrete into a high form was well illustrated. This particular concrete was a gravel concrete and the variation of the batch from sloppy wet to a consistency not more than plastic was made most evident. The impounding of air as such concretes go into the form was pointed out, with the bubbling and boiling of the material before settling into place. It was pointed out that free-flowing concrete need not be of sloppy consistency and that good concrete can be deposited by chutes with proper care; but the procedure as commonly carried out results in a decidedly inferior product, due to the very grossest carelessness and indifference with regard to these important particulars.

The operations of hand mixing were also shown, in which the dry materials were bedded in layers and cut down from the sides with shovels and hose. These motion pictures were most striking as illustrating utter indifference and carelessness in regard to consistency of mix that so commonly prevails. Hand mixing, as regards these matters, shows up even worse than machine mixing. Practices such as these might be characterized as permitted rather than as accepted practice; and a very wholesome lesson which should be heeded in every-day work was made most evident. It is not reasonable to expect concrete to be enduring and of value when the fundamental principles of proper manufacture are so commonly violated.

By showing the operations of cutting concrete with a blunt needle, these operations being taken through the microscope with the motion-picture camera, a direct presentation was had as to the relative qualities of wet and dry concretes and as to the relative values of concretes exposed to different classes of service. A concrete exposed to sea water was shown to be very soft and easily disrupted, being filled apparently with sulphates produced by the action of the sea water. It was very evident that the strength value of this concrete was exceedingly low and that its coherence was so uncertain as to make it almost valueless—conclusions borne out by its mass failure in service. This concrete had been exposed to waters in Long Island Sound for somewhat over four years.

On the other hand, a reinforced concrete beam was shown to have a fairly hard texture when dry; but this same concrete beam, when saturated with water, was shown to be quite mushy, to have a mealy texture, and to have its particles in such inchoate relation as to permit of their free movement about one another by the exertion of pressure or by poking them about with a pin—

visual illustration of some of the reasons why concrete takes a deformation under continued load. The contrast between this same concrete in wet and dry condition furnish a very striking illustration on the relative values of the average concrete with and without water exposure.

In like manner, the properties of mortars with standard sand, cured under standard conditions, were shown, together with contrasts of such mortars with field concretes as usually produced. Supplementing these motion-picture studies of texture, slides were shown illustrating the spherulation or individualizing of the cement particles through their dispersion by excess water, and their isolation through the formation of an impervious envelope about them by reason of their prompt reaction with the water. It was shown that it was quite possible and very probable that much of the relatively low strength of the average concrete is due to the dispersion of its cement particles to distances beyond their inter-crystallizing radii, so that they can properly unite to form a homogeneous bond.

To illustrate this point, a parallel was drawn between these dispersed cement particles and two individuals a mile or more apart, reaching out to clasp hands. It was pointed out that, bulk for bulk and distance for distance, the simile was not exaggerated; and that, if anything, the distance between individuals endeavoring to clasp hands and the distance between the cement particles endeavoring to inter-crystallize was in favor of the cement particles.

In other motion pictures taken through the microscope, the behavior of concretes in tension and compression was shown. The tension briquettes broken showed no deformation of the concrete before rupture. The briquettes crushed in compression, however, showed first a gradual flattening of the air voids and then a gradual crushing and flowing of the cement matrix in which the aggregate was imbedded. It was pointed out, as this action was flashed on the screen, that the cement constituent of concrete is the weakest element of the combination, both chemically and physically; that it is the most subject to attack; and that the less there is of it, consistent with proper surface covering and void filling, the stronger will be the concrete. The analogy between artificial concrete and natural concrete (*i. e.*, stone) was pointed out; and it was shown that the high strength of natural rock is very largely due to the exceedingly close compacting of the mineral particles, with a minimum of cementing substances between them.

In conclusion, it was pointed out that to improve the quality of concrete, the first step to be taken is to avoid those practices which are known to be detrimental. The second step is the production of improved mixing and placing as independent of the human element as possible. The third step is the proper selection and grading of the aggregates and constant watchfulness.

With the increase of proper knowledge in regard to the materials and making of concrete and the giving heed to the lessons taught, is bound to come to improvement in the quality and durability of this inherently excellent but much abused structural material.

THE PROPER USE OF CONCRETE GRAVITY CHUTES.

By W. H. Insley and C. C. Brown.*

The concrete gravity plant has had a very rapid development because of its undoubted economy in the time and labor cost of distributing concrete and its practically universal adaptability to all classes of concrete structures. The straight lift in a tower for the vertical distance between the mouth of the mixer and the top of the forms and then an additional lift of about one foot for every three of horizontal distance between the two before turning the concrete over to gravity to carry it across from the tower to the forms is about as near to Nature's absolute foot pound requirement as can well be devised.

Like any new process which, because of easily apparent advantages comes rapidly into general use, its use has outrun the rules of practice which a more conservative introduction would have established for it, with the result that every user has made his own rules with little guidance except his own experience and with as variable a product as this procedure might suggest. It is well, therefore, that some thought be given to the statement of some of the fundamental conditions which must obtain to insure a good concrete, which is of absolute importance, as well as to realize the largest ultimate factor of economy in operation, these two ends being obtained by the same means, the one depending on the other, the best concrete being the most economical to handle.

The typical plant consists of a tower with a hoist bucket which takes the batch of concrete from the mixer, a receiving hopper with a controllable gate, near the top of the tower, into which the batch is dumped from the hoist bucket, and a series of chutes or troughs which carry the concrete to the forms. The tower is frequently as high as two hundred feet and the line of chutes may carry the concrete as far as five hundred feet from the tower, and by using a relay tower the concrete is placed in the forms a thousand feet from the mixer. The chutes may be connected in a straight continuous line from the hopper to the forms, or this line may be interruped by line-gates through which the concrete is dropped a vertical distance through a closed pipe and then to the forms, or, by an assembly of horizontal swivel-connected chutes, it may travel in a more or less zig-zag path, dropping from the end of one chute into the swivel head of the chute below as it proceeds.

The matter of first importance to the successful operation of the gravity plant, as well as of any method of distribution, is the condition of the concrete when it is discharged from the mixer. Concrete is in proper condition for the gravity plant when it has been subjected to the action of a well designed mixer long enough to thoroughly incorporate all of the aggregates the batch being assembled with the proper amount of water to hold all of the aggregates in

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suspension, the resultant mixture being a viscous, homogeneous mass. As to how long the batch should stay in the mixer and as to the amount of water in percentages which this requires, our interest, so far as the chutes are concerned, must be confined to resultants and we must consider these questions as proper subjects for separate discussion. The concrete should not be so dry that it will not level off on top as it stands in the bucket, nor should it be wet enough to show water on top of the bucket if left standing for an appreciable length of time, nor to allow a larger stone to sink much over its own thickness when placed on top of the mass. Too dry concrete limits unnecessarily the range of distribution from a tower of a given height, by requiring a steeper chute to carry it. A wet concrete, which allows the heavier aggregates to settle to the bottom, will separate in travel and is to be avoided as one of the unpardonable sins. By all means, let the concrete be too dry rather than too wet, but there is the right consistency which avoids both extremes. But these problems are problems of mixing, however vitally they may affect the economy of the distributing plant. Properly assembled and well mixed concrete will maintain its integrity by whatever method it may be distributed, and concrete which is too wet will allow the stone to settle to the bottom of the form, and the mortar will come to the top, regardless of the means used to carry it there, while concrete properly assembled but too hastily mixed, will be very much improved by the movement through a line of chutes as against any other method of transportation. It must be borne in mind, however, that the gravity plant is a plant for distribution and not for mixing, and that the concrete must be good concrete, well mixed when it is delivered to the hoist bucket, or it cannot be expected to be good concrete when the forms are removed.

If the concrete reaches the chutes as a homogeneous mass, the slope of the chutes is not of vital importance. That slope is generally the best which will allow the concrete to flow with the least velocity which will insure its passage, although a vertical drop in a closed pipe is a feature of many installations on important work. Such vertical lines, however, should have baffles every few feet to arrest the drop, and the concrete should be distributed at the bottom by means of a horizontal chute, whenever possible, and not directly from the vertical line into the forms. The required minimum slope to carry the concrete properly will vary with the character of the aggregates, the average slope for small round gravel being one of rise to three of run, or an angle of about 18 degrees with the horizontal, the slope for 1-in. stone about 1 to $2\frac{3}{4}$, or 20 degrees, for $1\frac{1}{2}$-stone 1 to $2\frac{1}{2}$, or 22 degrees, and for 2-in. stone 1 to $2\frac{1}{4}$, or 24 degrees with the horizontal. It is better practice on a long line to hang the chutes with a gradually and very slightly increasing grade as they travel toward the lower end, such a grading being less likely to cause an overflow in the chutes than the reverse. The final distributing section, which places the concrete in the forms, should retard the concrete to as slow a movement as will carry it at all.

In the travel through the chutes, the concrete should flow in a constant uniform stream so far as possible. The man on the tower at the hopper gate is a very important member of the operating crew. An intermittent rush of

concrete is apt to congest the chutes, causing overflows, shut downs, the retaining of concrete in the tower hopper for an undesirable length of time, and damaged work.

The concrete should be placed by the chute as close as possible at the point where it is to remain. For floors and shallow beams the final chute section should be easily portable, with the mouth close to the forms and the concrete traveling as slowly as it can be made to run. For column forms and deep girders, the gravity plant provides a closed flexible drop pipe with frequent baffles or arresters for placing the concrete in the bottom of the forms, obviating the very objectionable practice of dropping it in the open from the top. If concrete is dropped from the top of a column form in the open or even in a closed pipe, without obstruction, the kinetic energy of the stones in the aggregate will drive them toward the bottom of the mass, separating them from the mortar; while if frequent baffles are placed in the vertical pipe the mass will retain its homogeneous character however deep the form may be.

To recapitulate, the greatest economy in operation is realized under the conditions which also disclose the best results in the character of the concrete when the forms are removed. The mixing crew must deliver to the bucket properly assembled and thoroughly mixed concrete of a viscous, homogeneous consistency. The chutes should be hung at as flat a grade as will readily carry this concrete, this grade varying with variations in the size and character of the aggregates. The concrete should flow through the chutes in a uniform, constant stream and should be placed by the chutes as close as possible to its final position, avoiding vertical drops without arresting baffles to neutralize the differences in kinetic energy of the aggregates within the mass. These conditions are primary, are easily realized on any work and will insure its success.

PRELIMINARY REPORT OF THE COMMITTEE ON REINFORCED CONCRETE BRIDGES AND CULVERTS.

In submitting its preliminary report, your committee has considered its purpose to be:

1. To promote interest and discussion of the subject.

2. To promote better design and construction methods and to eliminate failure in concrete highway bridge work.

All concrete bridge failures are due primarily to some one or a combination of three causes: 1. improper design; 2. poor materials; 3. bad workmanship.

To discuss these at all fully would require more time than is at the disposal of this committee. Moreover, investigations into concrete materials and methods of utilizing them are being made, not only by other committees of this Institute but by committees of other societies, for which reasons this committee has confined its report to a discussion of some of the problems relating to the design of concrete highway bridges. At some later date, doubtless, it will be advisable to supplement the reports on design with others bearing on materials and workmanship.

This report is comprised under four principal heads: Preliminary Investigations, Selection of Type of Structure, Bridge Loadings, and Details of Design.

Summary.

A summary of some of the more important conclusions which the Committee has reached, are as follows:

Preliminary Investigation.—1. The adequacy and correctness of a bridge design is vitally dependent on the character of the preliminary survey upon which it is based. Therefore, bridge designing departments should not send out plans for bridges based on the reports of unskilled local officials.

2. A study of the bridge site should be made only by skilled engineers or those specially trained for the work.

3. Bridge departments should issue blank forms, listing in detail all the information that the field engineer should secure.

4. The exact wording of the form is a matter to be decided upon by the individual organization, but the points discussed on the following pages, should be adequately covered.

5. A thorough knowledge of the behavior of streams is necessary before a proper report on the bridge site can be made. For this reason, bridge departments should issue information based on a study of the streams in the territory covered, for the guidance of field engineers, and should specially instruct the field engineer in regard to the importance of a thorough study of the stream in connection with the preliminary survey.

Selection of Type of Structure.—1. The selection of the type of structure is influenced by a large number of local conditions and can be properly made

only after a study of the same. In general, the following laws regarding selection, will hold.

(*a*) The slab is well adapted for spans of from 16 to 25 ft. where simplicity is desirable, and where the head room is low, for multiple span construction where low pier heights or foundation conditions render this type economical, and for locations near natural deposits of concrete materials.

(*b*) The through girder is adapted for spans of from 20 or 24 ft. up to 50 or 60 ft., where not to exceed 20 ft. roadways are required, where the head room is low, and where a massive architectural treatment is desired.

(*c*) The deck girder is adapted to spans of from 20 to 24 ft. up to 50 or 60 ft. where head room is not limited and to any width of roadway; especially adapted to viaduct construction where the greatest economy is desired. This type is susceptible of ornamental architectural treatment.

(*d*) The arch type is adapted for long-span construction, for construction over excessively high openings where the cost of abutments and piers for multiple short-span girders or slabs would be prohibitive, where clear spans longer than 50 or 60 ft. are necessary, and for construction carrying extra heavy traffic loadings. Arches are not to be recommended except for the very best of foundation conditions. This type is particularly adapted for locations where beauty of outline is demanded.

Bridge Loadings.—1. Considerable variation in stress may result from a variation in the weight of earth filling material, particularly in abutment and spandrel fill arch design. It is therefore recommended that for all important construction, accurate measurements to determine the weight of the filling material be made.

2. A knowledge of traffic conditions is necessary to determine the safe and economical load that a bridge is to carry.

3. Bridges should be classified according to the loads which they may be expected to carry.

4. A knowledge concerning the distribution of live load concentrations, through fills of varying character and depth, and also a knowledge of the proportion of such filling load to be transmitted vertically to the structure below, are points which vitally effect the correctness of the design. Experimental work looking toward the solution of these problems is under way at the present time. In view of the importance of these questions, it is recommended that, pending experimental data, the most conservative practice be adopted in this regard.

Details of Design.—1. The slab type is the simplest of all types in design, but a further investigation of the lateral distribution of load concentrations should be made. This problem is being investigated at a number of experimental stations.

2. The through girder type presents more complications in design. Further investigations are necessary to determine the position of the neutral axis for different conditions of loading, to determine the best distribution of the reinforcement and to settle other questions as discussed on a following page.

3. The deck girder type is more complicated in design than either of the foregoing; the chief point in doubt being the width of floor slab which may be considered as acting with the girder as a part of the T-beam. The safe value which may be assumed for this width must obviously be determined by further tests, but our present knowledge would recommend an assumed total width not to exceed 75 per cent of the distance center to center of girders, nor should it preferably exceed six times the width of the girder stem, nor eight times the thickness of the floor slab.

Special precaution should be taken to provide ample reinforcement for the girders, which reinforcement should preferably be proportioned to take the entire diagonal tension, assuming that none of this stress is borne by the concrete.

The reinforcement in the lower side of the floor should be made continuous over the girders.

4. In the design of arches, stresses arising from the lateral movement of foundations may greatly increase the stresses in the superstructure. For this reason, the use of this type should be restricted to those locations where the best foundations may be secured.

Reinforced arches should be proportioned to withstand stresses induced by a variation in temperature, given by the formula:

$$Y = 90° - 0.53\,X$$

where Y is the range in degree Fahrenheit and X may be taken as one half the average thickness of the arch ring. Where the yearly range of air temperature is less than 135° F., this value may be somewhat decreased.

The effect of axial compression or rib shortening may considerably modify the stress values and should be taken into account in the design of this type.

Reinforced arches should be provided with reinforcement throughout the entire arch ring, along both the intrados and extrados of the arch.

Vertical expansion joints are necessary where the spandrel walls join the abutment wing walls at the piers and at one or more points over the arch ring, depending on its length.

5. Substructure design. (*a*) Until further experimental data are available, the pressure to be assumed for back filling should not be less than that due to a fluid weighing 25 lb. per cu. ft. This value is to be increased if the filling is liable to be saturated.

(*b*) Special precautions should be taken to secure drainage of the back filling material.

(*c*) Bearing power tests of foundations should be made for all structures of any considerable size. In general, wherever piles can be readily driven, they should be used.

(*d*) If the character of the strata underlying the foundation is such as to necessitate the use of very short piles, the foundation should preferably be carried deeper rather than piled.

Part I.—Preliminary Investigation.

Incorrect and inadequate preliminary study in any type of engineering construction renders useless any nicety of design or detail. Especially is this true of construction in reinforced concrete, for structures of this material do not lend themselves to additions or alterations in design and can be wrecked only by incurring a tremendous labor cost with no appreciable salvage of material. Notwithstanding this apparent and obvious fact, your committee feels warranted in the statement that highway bridge and culvert work has had in the past a less amount spent for preliminary study than practically any other form of construction for public work.

Your committee has observed well organized designing departments working from field notes submitted by local, county, township, or municipal officials utterly inadequate in detail, and lacking in any information as to foundation material, stream behavior, etc. Even the simplest examples of sewerage, water supply or yard and terminal work are designed and constructed on information involving the work of a trained engineering corps for days and perhaps weeks, yet it is not uncommon for the county or other local officials to "measure up" their entire annual bridge work in one or two days.

The building of highway bridge structures (outside the limits of cities) in the majority of cases is, and in every case doubtless should be, controlled by a central organization whose jurisdiction should embrace not only design but preliminary engineering and construction as well. In the interest of efficiency, all of these functions should be centralized under one executive head, for the preliminary study, design, and execution of the work cannot efficiently be separated. It will often happen, however, even with such an organization, that the field engineering will be done by engineers who are not specialists in design. For this reason, it is very important to formulate definite rules and requirements for the preliminary field engineering, not only to eliminate the personal equation or variation in individual judgment but to insure that no detail is forgotten or neglected.

Your committee recommends that a standard form for bridge notes be furnished to the field organization. The exact wording and arrangement may well be left to the discretion of the central office but the following points should be covered:

I. Complete location data including:

(*a*) Names, letters, or numbers by which each bridge is to be designated. (The committee recommends the adoption of some adequate and simple method of bridge numbering as an undertaking well worth while where a central organization operates over any considerable territory.) The numbering of the bridges is obviously the province of the central organization and should be done under its direction according to a general system.

(*b*) County, township or parish numbers, or names, all section numbers, street or road names, etc.

(*c*) Local and official name of the waterway which the proposed structure is to bridge.

II. Designation of road or street, whether state, county or local road, and wherever possible complete notes on existing and probable future traffic conditions.

III. Distance from nearest shipping point.

IV. Source and cost of all concrete materials. This statement should include the length of haul from the source of each material to the bridge site and a description of the quality and availability of each material including stone, gravel, and sand. Your committee recommends that the field engineers, wherever the plan is feasible, be required to submit to the central office samples of the materials available and that the forms be worded so as to call attention to this requirement.

V. Prevailing price of labor and teams, and average net weight of load which may be hauled with one team from source of supply to bridge site.

VI. Drainage area in acres.

VII. Character of watershed, whether gently rolling, rolling, hilly, very hilly, mountainous, or very mountainous; whether all or in part in a state of cultivation, etc.

VIII. Width and depth of stream at ordinary water stage.

IX. Width and depth of stream at flood stage.

X. Additional information in regard to the area of waterway required. With the drainage area and character of the watershed known, it becomes necessary to apply one of a dozen or more empirical formulas to determine the probable needed waterway.

It is quite generally accepted that all formulas for flood flows are more or less inaccurate except for one given set of conditions. Especially is this true of those formulas which do not involve the slope, intensity of rainfall and character of the surface. Probably the most accurate method in proportioning waterways is by comparison with neighboring structures which have practically the same drainage area, providing a waterway equal to a known waterway which has proven adequate, adding a certain percentage as a margin of safety, to be decided upon from the local conditions. For the above and other reasons, data concerning the size and dimensions of the old structure, its adequacy in high-water periods, and wherever possible the structure immediately above and below it are well worth the trouble.

Your committee would, therefore, commend the practice of requiring the field engineers to prepare and submit sketches of structures immediately adjacent to the proposed structure.

It is advisable in any case to use a waterway formula for preliminary estimating and as a check on the above. Such a formula plotted in the form of a chart, preferably taking into account the slope, rainfall intensity, and character of surface, will doubtless prove of much value to field engineers.

XI. Description and elevation of all bench marks. A permanent bench mark or bench marks should be established at each bridge site by the survey party and a record of the same made a part of the bridge notes.

XII. Specific elevations. In order to insure that no detail is missing, certain elevations should be called for on the form furnished the field engineers. Among them:

(*a*) Elevation of highest water mark.

(*b*) Elevation of ordinary high water.

(*c*) Elevation of low water. This elevation is of importance as fixing an elevation below which no attempt need be made to hide construction joints.

(*d*) Elevation of highest ice mark, which is very important in pier design and in open spandrel arch construction.

(*e*) Elevation of lowest point in channel.

XIII. Relief openings for flood water. It often happens that in flood times natural or artificial relief openings along the roadway will assist in carrying the flood water. The roadway grade elevation located within the flood plane limits may be lower at some point than the clearance of the new structure, in which case, when the new structure runs full, considerable flood water will be taken care of. If the road grade is permanent (will never be raised), this fact may be utilized to reduce the size of opening and the consequent cost.

Your committee recommends, therefore, that the following be inserted in the field note form:

"Will all flood water pass through new structure?—(Illustrate by sketch and explanation in full, giving all elevations referred to established B. M.)."

XIV. Data as to possibility of clearing channel to afford more waterway. It not infrequently happens that a little money spent in channel work may considerably reduce the cost of the bridge structure. Your committee recommends the following insertion in the field note form:

"Can channel be cleared to afford new waterway? (Illustrate by sketches in full)."

XV. Stability of stream in its banks. Streams in the third or old age stage of development, having cut to their base level, are, in general, more or less unstable in their banks. Not infrequently otherwise permanent construction may be jeopardized by a diversion of the channel toward or against one abutment. For this and other reasons, it seems the part of wisdom to insert the following question in the field note forms:

"Is main channel shifting laterally in either direction? If so, indicate by ample sketches."

XVI. Stream behavior and characteristics. Probably no other factor so vitally effects the life of our short-span highway bridges. That over ninety per cent of the failures in short span construction are foundation failures and that by far the majority of these are due to erosion, is undoubtedly a conservative estimate.

For streams not yet to their permanent grade or base level, the cutting capacity varies theoretically as the square of the velocity while the carrying capacity (which in alluvial soils largely determines or limits the erosion) varies theoretically as the velocity to the sixth power.* The velocity in turn is a direct function of the volume and of the gradient and an inverse function of the load or amount of eroded material carried in suspension.

* The values here given are theoretical values and are of course modified by local stream conditions. For a complete discussion of the phenomena of stream flow, see Professional Paper No. 86, U. S. Geological Survey, entitled "The Transportation of Debris by Running Waters," by G. K. Gilbert.

As the stream approaches a permanent grade or base level, an equilibrium is established and the eroding action ceases.

These fundamental laws of stream development not only emphasize the tremendous erosive action that may accompany a sudden high water in cutting streams, but also that any external agency operating to change the general stream gradient will operate to increase the velocity and consequently the erosive action. Your committee has observed a channel change transform a filling mature stream into an eroding ditch with consequences disastrous to foundation work.

Fortunately, although the ultimate base level of a waterway is dependent upon conditions covering such a wide topography as to preclude careful study, the general elevation to which any stream will erode may be approximately estimated from the profile of the stream bed above and below the bridge site for a distance sufficient to include the various factors affecting the velocity. The general tendency where the material is of like nature throughout the length is to establish a uniform grade. However, this law is modified by so many factors that no definite law may be laid down.

Your committee would recommend the distribution of condensed information concerning stream behavior over the territory covered, to all field corps, and that in the form for preliminary survey the following points be covered:

(*a*) Is the stream at present, cutting or filling?

(*b*) What is the character of the strata through which the channel has cut?

(*c*) What is the average fall in feet per mile?

(*d*) Will any future channel change increase the gradient?

(*e*) Submit profile of channel (not cross-section) for— ft. above and below bridge site.

(*f*) Is there any liability of local scour due to a sharp angle in the course of the stream or the lodgment of drift. Submit sketches.

XVII. Foundation data. The safe bearing power of the soil supporting the foundations is generally assumed from a physical examination of the material by means of test pits, soundings, etc., although several devices to measure the yielding under various unit loadings (and thus the safe unit loading of the foundation material directly) have been used with more or less success. The committee commends this practice in important structures. But whether such a device is used or not, careful soundings should be made and a place for such data included in the blank form furnished for this purpose.

It may be well to require the field engineer to submit a recommendation as to the feasibility of driving piling, together with data as to the minimum length of pile needed, etc. In general, if the foundation material is such that piling of any considerable length can be driven the same are necessary. Especially is this true on cutting streams.

XVIII. Data regarding old bridge. These should include: 1. type; 2. number and length of spans; 3. width of roadway; 4. availability of any or all of the old material for the new construction (temporary bridge, etc.); 5. sketch showing area of waterway afforded by old structure, alignment of the structure with reference to the road and to the stream, skew angles, etc.;

6. a profile of the old floor together with a portion of the road grade on each side. The length of the profile will depend upon local conditions, but should be sufficient to furnish data for the establishment of a new grade if needed and for computing the areas of any relief waterway in times of high water.

XIX. Recommendations for new structures. The field engineer, being conversant with the problem, should be competent to submit the best recommendations for the new structure. This should include: 1. type; 2. number and length of span; 3. roadway widths; 4. data as to sidewalks, ornamentation desired, etc., and complete sketches showing location of new structure with reference to the old, both in elevation and plan; 5. ground elevations or contours sufficient to enable an accurate computation of excavation, etc., and also to furnish data for the wing wall designs.

XX. Additional information. On the sketch prepared to show the new bridge should be given information from which the permanent bank lines of the stream can be platted and in addition, an accurate cross-section of the waterway. Skew angles, recommended angles and elevations for all wing walls, proposed method of taking care of the drainage water from the road side ditches, etc., should also be included.

In concluding this subject, your committee wishes to point out that the considerations are not elementary but of the most vital importance in the development of standard methods for a bridge organization.

It has been the intention to cite only the principal points to be covered and undoubtedly many other points varying with the particular needs involved should be covered in any standard form or manual of instructions to surveyors and field engineering corps.

PART II.—SELECTION OF TYPE OF STRUCTURE.

The selection of the type of construction best suited to the needs of the location is a problem considerably involved. The type of structure is influenced by many factors, among them being: 1. the availability of concrete materials; 2. the haul of materials; 3. available head or clearance room; 4. traffic conditions; 5. architectural considerations; 6. field supervision available; 7. character of foundation material; 8. stream conditions as regards erosion; 9. temperature conditions, and many others.

Each of the more common types of construction possesses distinct advantages and disadvantages, and has its place in the economy of design. A brief resumé of each type follows:

SUPERSTRUCTURE.

The Slab.—Designed as a simple beam, this type possesses the advantages of:

A. Simplicity of construction.

1. The form work is comparatively simple, requiring less cutting of lumber than in any other type.

2. The reinforcing in general (especially for the shorter spans) may be of bars smaller than for most of the other types, admitting of easy bending

in the field. The reinforcement system is not complicated and is so placed as to be readily accessible and in plain sight, so that errors in alignment may be corrected immediately before the concrete is poured.

B. The dead load of green concrete is uniformly distributed over the falsework and the expense of providing for a support for concentrated loadings (as in the case of the girder types) is reduced.

C. The sections are relatively massive. Even our best stone construction is affected by atmospheric agencies such as temperature, moisture change, etc. This action is also observed in concrete construction and is manifested by a spalling at corners and angles and the appearance of small cracks on the surface. Obviously a smaller proportion of mass concrete will be thus affected. It is for this reason to be preferred to thin section work.

D. The type admits of a future widening of roadway without impairing the strength of the existing structure.

The principal disadvantage is in the excess of material required. Since the strength of a beam varies as the square of the depth and first power of the width, this type can never compete, yard for yard, with the ribbed or girder type.

The slab type is admirably suited, (1) for local construction where simplicity is essential; (2) for spans up to about 20 to 24 ft., where the reduction in cost for forms and falsework, labor, etc., will doubtless offset that due to the increased yardage; (3) for multiple span construction where low pier heights or foundation conditions render the same economical; (4) for locations near a supply of concrete materials, and (5) for locations where there is a possibility of a future widening of roadway.

The Through Girder Type.—The principal advantages of this type are:

A. As compared with the slab type, the amount of material is less.

B. As compared with the deck girder type, the sections are relatively massive and are, therefore, to be preferred for the reasons given above.

C. As compared with either of the above types, the head room, or clearance occupied is less.

Its principal disadvantages are:

A. As compared with the slab type the form work and reinforcing are more expensive and complicated, requiring more skill to construct, and the dead load of green concrete is concentrated on the falsework underneath the girder, entailing more expense for this item of construction.

B. The construction does not admit of a widening of the roadway to accommodate increased traffic requirements.

C. The rails are of necessity massive and thus the type does not lend itself to ornamentation as readily as do the other types, where spindle balustrades, ornamental posts, etc., may be employed.

The Deck Girder.—Owing to the utilization of the floor slab as the compression flange of the girder T-beam, this type can be constructed with less yardage than either of the other types. This constitutes the principal advantage of this type, although among the other features to recommend it are:

A. Its adaptability to different schemes of ornamentation to suit the particular needs of the location.

B. The fact that it can (though with difficulty) be widened to accommodate increased traffic requirements.

Its principal disadvantages are:

A. The relatively light sections of floor slab that are affected to a greater extent by the general weathering of the material, as above discussed.

B. The relatively great head room or clearance occupied.

C. As compared with the slab type, the reinforcement and form work are more complicated and the skill required to construct is greater.

The place in the economy of design for the deck girder type is in single span construction in lengths between 20 or 24 ft. and 50 or 60 ft., and for multiple spans of the same lengths. The through girder may be constructed in single or multiple spans of these lengths, but the roadway, especially for the longer spans will, for reasons of economy, be limited to 20 ft.

The deck girder type lends itself most readily to construction over relatively deep ravines, to localities where materials for concrete are scarce and the hauling cost great, and for locations where a light and ornate architectural treatment is desired. The through type readily recommends itself for low head room construction, and for construction requiring massive architectural treatment.

The T-Beam.—This type of construction consists in the use of small beams, spaced closer together than in the deck girder type, and covered with a thin floor slab which furnishes the compression area of the girder T-beams. This type is an adaptation of the deck girder type. While the yardage is less, the sections are much thinner than the slab type, and, if chosen, this type should undoubtedly be subjected to the most rigid inspection, and constructed with carefully selected materials.

The Arch.—Used in single or multiple spans from 40 or 50 ft. up, the fixed arch has to recommend it:

A. Its beauty of line and the great variety of architectural expression possible.

B. The fact that the live loading stress (except in open spandrel construction) is a very small percentage of the total stress, thus minimizing the danger of serious over-stress from increased traffic loadings.

C. The ease with which the type may be widened to accommodate future needs.

Its principal disadvantages are:

A. The high cost of form work and centering, the difficulty in placing and holding the reinforcement and the difficulty in holding the concrete on the steep incline at the haunch necessitating back forms in many cases, which are expensive.

B. That uneven foundation settlement or a slight spreading of abutments may seriously overstress the superstructure. This last is most important in view of the amazing number of light arches which have badly cracked and, in many cases, totally failed from this cause.

The arch type is suitable for long-span construction, for construction over excessively high openings when the cost of abutments and piers for short span girders or slabs would be prohibitive, where clear spans longer

than 50 or 60 ft. are necessary, for construction carrying heavy city loading and where considerable expenditure is warranted for artistic treatment. The type, however, is only suitable where the natural foundation conditions *are of the very best or where funds are available for artificially rendering the foundation absolutely safe against unequal settlement, or lateral movement.*

The three-hinge arch has been used to a slight extent both in the United States and in Europe, notably the hinged Melan construction, such as the bridges at Steyr and Laibach in Austria, and the hinged "unit construction" known as the Thomas system and used to a more or less extent in California and vicinity. This type has not come into extensive use.

Substructure.—In practically every form of abutment construction, the overturning forces, consisting of the lateral components of the pressure of the retained material, are resisted by two forces: (1). the weight of the material in the substructure proper, and (2). the weight of that portion of the retained material placed over the rear footing slab.

This fact has been recognized since as early as 1869 and the greater proportion of substructures, whether plain or reinforced, have been designed with footings. There are two principal types of abutment substructures in common use for highway bridge work.

The Plain or Gravity Type.—This type derives its name from the fact that it is comparatively massive, and the weight of the retained material is a smaller proportion of the total weight than in the other types. The form work is very simple and the dimensions are, in general, sufficient to permit the use of larger stone (hand stone) in the concrete. Where concrete materials are at hand or where quantities of stone are available for cyclopean work, this type recommends itself.

The Reinforced or Cantilever Type.—This type has simply been adapted from the former type by the substitution of a lighter reinforced section for the heavier unreinforced section there used. It is, therefore, more economical of concrete but more expensive as regards form work, reinforcing and labor to construct.

When the wings are rigidly anchored to the abutment body, the entire mass may be made effective in resisting overturning. This results in still further economy of concrete without increasing the cost of form work or labor.

As ordinarily constructed, the vertical stem of this type of construction acts as a reinforced concrete cantilever transmitting stresses to the footing slab directly. A modification of this type consists in constructing a ribbed stem wherein the stresses are first transmitted laterally from a thin slab to vertical ribs or counterforts and thence vertically to the footing slab. This modification results in a saving in concrete but entails additional expense for form work and labor. For heights above 30 ft., the saving in material thus effected may more than offset the additional cost for labor and form work.

The Reinforced Concrete Trestle or Viaduct.—This term has been applied to a construction whose superstructure consists of multiple span slabs or deck girders, and whose substructure design consists of reinforced bents (generally two post bents with horizontal collar bracing, caps and sills) for piers; and braced bents forming a tower through which the earth takes its natural slope for abutments.

This type of construction is undoubtedly cheap in first cost, especially over deep and long ravines, but your committee believes that such construction should be used sparingly and only after the most rigid analysis of design. Concrete as a structural material lends itself to mass construction and any attempt to lighten up the general scheme of design should be well considered. Your committee believes that construction such as this has a place in the economy of concrete bridge construction, but only when conditions of stream erosion, stability of stream banks, danger from flooded débris, and ice have been fully taken into account and properly provided for.

PART III.—BRIDGE LOADINGS.

Dead Load.—For important construction, or that requiring a nicety of design, field measurements of the actual unit weights of the materials to be used may be warranted. The ordinary structural materials will not vary materially in unit weight. For filling material under different conditions of drainage, consolidation, etc., the unit weights are not so uniform. Thus the weight of earth filling may vary by more than 40 per cent. By means of small brass cylinders carefully driven into the material in situ, data have been collected concerning the maximum density of earth filling under varying conditions. The range of unit weights which may be expected is illustrated by Table I from results of actual measurements as above described:

TABLE I.—WEIGHTS PER CUBIC FOOT OF SOLID UNDISTURBED SOIL IN PLACE.

Kind of Material.	Damp.	Saturated.
Black top soil	94	105
Yellow clay	121	123
Sandy clay	106	115
Clayey sand	123	132
Blue clay	114	118

In view of the fact that a difference in weight may affect the design to a considerable degree (particularly in abutment and spandrel fill arch design) your committee recommends for all important construction accurate measurements to determine the unit weights of filling material.

In the absence of accurate experimental data for the particular case involved, the following unit weights may be taken as representing average values.

MATERIALS.	LB. PER CU. FT.
Earth filling	120
Plain concrete	145
Reinforced concrete	150
Steel	490
Cast iron	450
Vitrified brick	140
Common brick	125
Granite and limestone masonry	165

Materials.	Lb. per Cu. Ft.
Sandstone	140
Macadam-Telford	150
Pine, fir, etc	42
Oak and yellow pine	48
Creosoted timber	60

Live Loadings.—Conditions governing the selection of a standard live loading over a territory of the size occupied by the membership of this Institute are so widely varied that to design for a loading which in one locality would be no more than safe practice would result in extravagance for less populous localities.

The live load which, in the interest of both safety and economy of first cost, should be selected is that loading to which the bridge structure during its estimated life is likely to be ordinarily subjected. This involves not only a study of the traffic conditions of today, but a prediction of future development, which latter can be based only on a careful study of past growth. To emphasize the magnitude of this question and the tremendous waste of funds which might easily accrue to the taxpayers of the United States through an improper selection of loadings for their highway bridges would be superfluous.

It is hoped that in the near future, through correspondence with manufacturing firms and street railway officials, and through co-operation with the engineering profession in general, more data looking to the selection of proper loading assumptions may be collected by this committee.

After a consideration of the data at hand, the following recommendations are offered pending further investigation.

Class A Bridges.—City bridges and bridges on main thoroughfares leading therefrom.

Concentrated Live Load.—A motor truck of the following dimensions and weights:

Total weight	50,000 pounds
Weight on rear axle	33,000 pounds
Distance between axles	10 feet
Width of tread of rear wheels	24 inches
Distance between centers of rear wheels	6 feet
Roadway space occupied, width	10 feet
Length	30 to 36 feet

For city bridges in the areas of heavy traffic (especially if a crossing at either end of the span operates to hold up traffic), it not infrequently happens that trucks of this character follow one another so closely as to constitute a connected string reaching more than across any ordinary single span. Outside the limits of the heavy traffic areas and on the tributary main highways, the possibility of more than two trucks being upon the span at any one time is exceedingly remote.

It is, therefore, recommended that for the former locations as many of the above motor trucks as are necessary for maximum stresses be considered

on the structure at one time; for the latter locations the number may in all safety be limited to two.

The following alternate uniform live loadings are recommended:

Span, ft.	under 80	80–100	100–125	125–150	150–200	over 200
Loads, lb. per sq. ft.	125	110	100	90	85	80

It is further recommended that the above uniform live loading be assumed to occupy all or such portion of the roadway area not occupied by the motor truck loading as is necessary for maximum stresses.

Sidewalk Loadings.—The greatest legitimate loading likely to be imposed upon a bridge sidewalk will rarely exceed 90 lb. per sq. ft. of area.

In this connection it is recommended (especially for city bridges) that special precaution be taken to protect the sidewalk area from any of the street traffic. If the details of the design are such that this cannot be done, it is recommended that the sidewalk area be designed for the maximum street traffic.

Class B Bridges.—The minimum requirements for ordinary highway traffic should probably be within the following limits:

Concentrated Live Load.—A traction engine or motor truck within the following limits:

Total weight	30,000–36,000 pounds
Concentration on rear wheels	$66\frac{2}{3}$ per cent
Distance between axles	10–12 feet
Distance between rear wheels	$5–6\frac{1}{2}$ feet
Width of tread of rear wheels	20–24 inches

Only one such vehicle need be considered upon any single span at one time. The floor area occupied by the above live loading may be taken as 10 ft. in width.

The alternate uniform live loading on Class B spans of varying lengths may safely be assumed as follows:

Span, ft.	under 80	80–100	100–125	125–150	150–200	over 200
Load, lb. per sq. ft.	100	90	80	75	65	60

Class E-1.—Bridges carrying *ordinary* electric railway traffic. It is recommended that these bridges be designed to carry a concentrated live loading of the following dimensions and weights:

Total weight	100,000 pounds
Number of wheels (on two trucks)	8
Spacing of wheels, center to center	7 feet
Spacing of trucks, center to center	20 feet

Each axle load distributed over three ties.

Special Bridges.—The electric railway loading above specified will doubtless be exceeded by the heaviest interurban traffic and by heavy electric

engines carrying freight, which latter may exceed 100 tons. These, together with loads resulting from traffic from mines, quarries, manufacturies, etc., all constitute a special class and bridges carrying same should, of course, be designed to meet local present and future requirements.

Distribution Through Fills.—The distribution of live load concentrations through fills of varying character and depth, and the proportion of such filling load to be transmitted vertically to the structure, are questions concerning which few data are available. Experimental work looking to the solution of these problems is now under way at the University of Illinois Engineering Experiment Station and some very excellent work along this line has been done by the State College of Pennsylvania. The United States Office of Public Roads and Rural Engineering, Washington, D. C., is also conducting an extensive series of tests along this line, and it is hoped that in

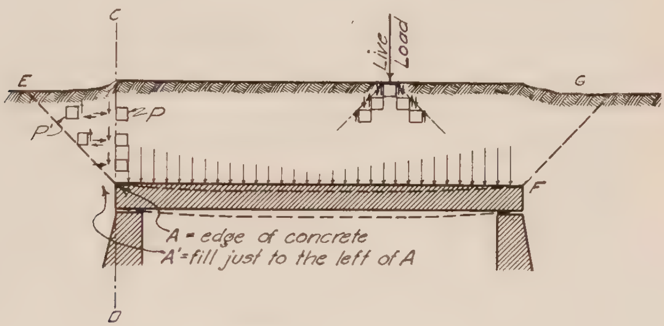

FIG. 1.

a future report definite recommendations based on the results of the above mentioned and other experimental work may be submitted.

A study of the theory underlying the transmission of pressure through fills furnishes data indicating that the entire weight of the filling material placed over a superstructure, either arched or flat, may, under certain conditions, be transmitted vertically to the same, and that for flat top construction a portion of the filling material at either end of the span may be transmitted to the span furnishing a small additional load.*

* That this condition may in practice occur may be seen from an inspection of Fig. 1, which may represent a slab under a newly constructed fill. Each particle of the filling material is under a vertical pressure which induces a lateral pressure and frictional stress between it and its neighbor as indicated by the arrows along the line *CD*. The particles which move the greatest distance will obviously be restrained to a certain degree by the adjacent particles and will consequently transmit a portion of their load to the same. At *CD*, therefore, unless the concrete at *A* deflects more than the fill at *A′* (which is inconceivable under the conditions here outlined) not only will the entire weight over the slab come directly on the same but a portion of the vertical pressure on the particles *P′* outside the span limits will be transmitted by friction to the span, so that the effective load may be a trapezoid *EAFG*. Conditions may be such as to reverse the direction of the friction stresses and decrease rather than increase the filling load, but the fact that under certain conditions the first outlined condition may obtain seems enough to warrant the tentative recommendation here given.

Until further data are available, therefore, it seems the part of prudence to recommend for both arch and flat top construction that at least the entire weight of filling material included between two vertical planes a distance apart equal to the clear span be considered as bearing directly upon the superstructure.

The pressure underneath a concentrated live load, on the other hand, will be distributed through the fill, the amount of such distribution varying with the depth and character of the filling material. It is hoped that a definite recommendation as to the amount of such distributing action may be made in a future report.

Expansion and Contraction.—In the design of flat top bridges, if the abutments are assumed to be rigid and the superstructure is made integral therewith, a lowering of temperature of t deg. will induce tensile stresses given by the formula:

$$S = LcE$$

where S = the unit tensile stress pounds per square inch,
c = the coefficient of thermal expansion,
E = the modulus of elasticity.

Substituting in this formula a temperature drop of 40° Fahrenheit would set up in the reinforcing steel tensile stresses of 7200 lb. per sq. in.

For this reason your committee recommends that provision be made for expansion and contraction, either by the introduction of an expansion joint or by proportioning the reinforcement to resist the stresses thus induced.

Impact Stresses.—The subject of impact stresses will be taken up in a later report.

Unit Stresses.—The maximum allowable unit stresses in reinforced concrete and in reinforcement should conform to the recommendations of the Joint Committee.

PART IV.—DETAILS OF DESIGN.

The Slab Type.—This is designed as a simple beam and reinforced for main and diagonal tension. This type presents fewer problems to the designer than any other.

The principal detail of design concerning which present knowledge is meager, is the amount of lateral distribution of load concentrations.

Experiments by the United States Office of Public Roads and Rural Engineering, Washington, D. C., made on a slab of 16 ft. span and 32 ft. width, 12 in. in thickness and reinforced with ¾-in. square bars 7¼ in. center to center were discussed in a former number of the *Journal** of this Institute. These experiments indicate, for concentrated loadings of from 25,000 to 32,500 lb., a distribution over an effective width of seven-tenths of the span. No transverse reinforcement was used.

Experiments by the Engineering Experiment Station of the Iowa State College, as yet incomplete, include the testing of a 12-in. slab, 18 x 16 ft., reinforced with ¾-in. square bars 6 in. center to center longitudinally and

*See p. 117, No. 2, Vol. III.

½-in. square bars 12 in. center to center transversely. For a loading of 23 tons placed on two tractor wheels symmetrical about the longitudinal center line of the bridge and at the center of span, the wheels being 6 ft. center to center and 24 in. in width, the strain gage readings indicated a longitudinal stress nearly uniform throughout the entire width.

Further experimental data looking toward the development of the underlying theory of stress distribution in flat slabs are badly needed, and undoubtedly the continuation of these and other tests along the same line will furnish information of considerable value. In view of such experimental data as are at present available, it seems safe to recommend a transverse distribution of concentrated loadings over a distance of from one-half to two-thirds of the span lengths for spans such as are ordinarily constructed of this type and that all transverse reinforcement be proportioned accordingly.

The Through Girder Type.—In this type of construction, the distribution of stress is affected by a greater number of factors.

First.—The suspended floor is a beam partially fixed at the ends. The degree of this restraint depends on the relative resistance of the girders against deflecting laterally inwards. Under concentrated loadings occupying but a small percentage of the total span length, such lateral deflection would be materially less than with a uniform live load over the entire span.

Second.—The position of the neutral axis in the girders is modified to a large extent by both the concrete and the longitudinal steel in the floor.

Experiments conducted by the Illinois Highway Commission on a 40-ft. span of this type located at West Chester, Ill., furnish data such as to indicate:

1. A progressive rising of the neutral axis with load increments, which is in accordance with theory.

2. That the mean observed position of neutral axis (which represented a load of 150 tons on the girder) was but 1 in. above the computed position based on an uncracked section including the floor.

3. That the action of the suspended floor was such as to indicate a point of contraflexure 9 in. distant from the face of the girder, while the computed point of contraflexure, based on an assumption of entire fixity, was 3 ft. 6 in., thus denoting an inward tilting of the girders under uniform loading.

The effect of the side girders in restraining the suspended floor under concentrated loadings, and the amount of restraint for varying sizes of girder and width of roadway, are points concerning which there is need of more experimental data. The relative efficiency of the longitudinal steel in floor and girder should be investigated further, as considerable economy might result from a further lowering of the neutral axis.

Among the other points of design concerning which experimental data are much needed, mention may be made of the following:

1. The relative effectiveness and efficacy of vertical stirrups, bent up diagonal rods, and combinations of both in preventing diagonal tension cracking.

2. The distribution of load concentration on the suspended slab and the evaluation of the longitudinal stiffness factor in transferring floor loads directly to the abutments.

The Deck Girder Type.—Aside from the arch, this type presents more problems to the designer than any herein considered.

When the floor is independent of the supporting girders, the action is that of a beam continuous over a series of elastic supports. A rigid calculation of stresses is then possible only by the method of least work or by considering all but two girder reactions redundant. However, in practice a portion of the floor slab is utilized as compressive area for the girder, the latter being figured as a T-beam. The width of the floor slab thus effective and consequently the deflection of the girder under unit loading, cannot be determined from the dimensions of the design. This so complicates the problem that any laborious calculation is unwarranted.

The location of the girder neutral axis will depend upon the width of slab which is effective as T-beam compressive area. Experimental data tending to clear up this point are noticably lacking and, in view of the fact that the entire design of the girder system depends thereon, are much needed.

The greatest problem is the one of distribution at right angles to the line of stress of load concentrations on the floor slab. This not only affects the design of the floor slab itself but modifies the girder loading.

The extent to which the girders operate to fix the ends of the floor slab, the effect of combined T-beam and floor slab stresses at right angles to each other, these and many other points emphasize the need for a great amount of research along this line.

In view of the lack of such data, your committee can only submit the following tentative recommendation as to designing practice:

1. That in view of the fact that the action of the floor is undoubtedly analagous to that of a continuous beam over multiple elastic supports, the bottom reinforcing be made continuous over the girders, thus being effective to distribute any concentration to adjacent girders.

2. That no such distributing action be considered in proportioning the girders but that the latter be proportioned for the entire load immediately over them plus the static moment reaction of the loadings on adjacent floor slabs.

3. That in view of the high web stresses which arise in the girder web of this type, the web reinforcement be proportioned with special care. Both stirrups and bent up rods should preferably be used and great care taken to avoid any possibility of bond failure.

If the concrete is considered as taking diagonal tension, the unit stress in the same should be proportioned on the basis and limited to the value recommended by the Joint Committee. Preferably for bridge work, the web reinforcement should be proportioned for the entire shear, no dependence being placed on the concrete.

4. The width of flange which can, in good practice, be assumed as effective compression area may be governed by the limitations of the Joint Committee. For bridge work, it seems advisable to limit this width still further, as indicated in Fig. 2. If possible, a less width than the maximum there shown should be assumed.

Data upon which to base any assumption concerning the distribution

(at right angles to the line of principal stress) of concentrated loadings on thin slabs of the character of the floor construction in this type, are as yet somewhat meager, although some excellent experimental work has been done by the University of Illinois and the Ohio State Highway Department. The experiments of Slater (University of Illinois) were on slabs of 3 in. and 6 in. depth, in spans principally of 48 in. and of widths varying from 24 to 96 in. The principal value of the preliminary treatment is in the development of theory and as showing the value of transverse reinforcement. The data published by the Ohio State Department (Bulletin No. 28) were secured from the testing of $3\frac{1}{2}$ ft., 5 ft. and 7 ft. spans of 7-in. and 4-in. thickness and of varying widths. From these tests, the equation $E=0.6S+1.7$, (where E is the effective width over which a load concentration may be considered as distributed and S the clear span) has been developed. The result of the

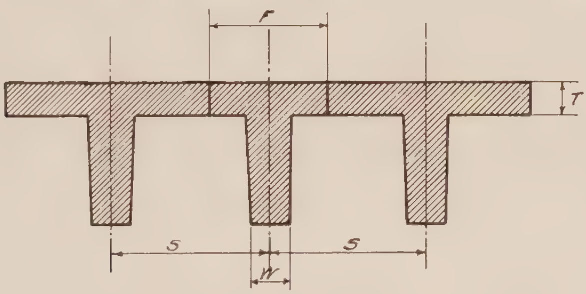

FIG. 2. *F* IN GOOD PRACTICE SHOULD NOT EXCEED
a, 75 PER CENT *S*.
b, 6 TIMES *W*.
c, 8 TIMES *T*.

U. S. Department test alluded to on a former page,* seems to fall very close to the locus of this equation.

The data from the Ohio Department tests indicated a distribution independent of transverse reinforcement while the experiments of Slater indicate that such is not the case. More data are needed looking to the development of formulas, etc.

Substructure.—The correct design for abutments and wing walls requires a knowledge concerning the amount, direction and point of application of pressure of the retained material. Concerning this, very little is actually known, although a great deal of theoretical work along the line of analysis forces has been published.

The formula of Rankine, probably more often quoted than any other, is as follows for vertical walls:

$$P=V\left[\frac{\cos^2\phi}{\cos^2 d\sqrt{\dfrac{1+\sin(\phi+d)\ (\sin\phi-d)^2}{\cos d^2}}}\right]$$

*See p. 416.

where ϕ = angle of repose of the material,
V = vertical pressure,
d = angle of surface with the horizontal,
P = pressure parallel to the surface.

For material that will not stand at a natural slope, steeper than one and one-half to one, the formula results (for horizontal surfaces) in a horizontal pressure equal to about one-third the vertical. For materials which will stand at a one to one slope, the horizontal pressure is reduced to about one-sixth the vertical.

For varying weights and conditions of back filling, the horizontal pressure is thus seen to vary from an equivalent fluid pressure of 15 lb. to one of 50 lb. per sq. ft.

Experimental data along this line are much needed and it is hoped that such data may be secured at a future date by your committee.

In ordinary practice for well drained back filling, it is not probable that a horizontal pressure much higher than that of a fluid weighing 25 lb. per cu. ft. will be imposed. For clayey soils, however, this may be greatly increased and may, in fact, assume any value up to hydrostatic pressure.

The action of frost also may operate to exert pressure considerably in excess of the above limits.

For the foregoing reasons, it is recommended that, in the absence of further experimental data:

1. Abutments, wing and retaining walls under the most favorable conditions of drainage, and natural stability of back filling material, be designed to withstand a pressure equivalent to a fluid having a unit weight of not less than 25 lb. per cu. ft., and that this value be increased for all materials or any location where saturation of the back filling, slope slides or poor drainage conditions are likely to be encountered.

2. Special precaution be taken to secure the best possible drainage of the back filling material.

In construction of this character, the overturning moment and consequently the toe pressure on the foundation may be greatly diminished by utilizing the weight of the attached wing walls. When this is done, care must be taken to provide special reinforcing at the junction of wing and abutment body, so that the two may act as a monolith. That this point has been lost sight of, is evidenced by the great number of smaller bridge structures disclosing a rupture of the wing walls at their junction with the main abutment body.

Your committee, for this reason, commends the practice of placing near the top of and across the plane where the body of abutment and wing are joined, sufficient steel to resist the entire horizontal earth pressure on the main wall of the abutment. It may also be advisable to unite the bases of the abutment body and wing by reinforcing.

Of the three conditions which limit the design of abutments and retaining walls, viz:—stability against overturning, stability against sliding along the base and maximum allowable toe pressure, the last most frequently enters

the problem as the determining factor. The determination of the safe values for different soil conditions is then a matter of prime necessity.

The subject is under investigation at the present time by one of the committees of the American Society of Civil Engineers. Pending this report and further investigation of the subject, your committee wishes to submit the following tentative recommendations for allowable bearing values of various foundation materials:

MATERIAL.	SAFE BEARING POWER, tons per sq. ft.
Quicksands and wet soils	0.1 to 1.0
Dry earth, according to depth below surface	1 to 3
Moderately dry clay confined	2 to 4
Dry stiff clay	4 to 6
Sand confined	2 to 6
Sand compact and cemented	4 to 8
Gravel cemented	8 to 12
Rock	25 to 200

For structures of sufficient importance to warrant such a procedure, it is recommended that the bearing power of the soil be determined by actual tests.

A device such as is shown in Fig. 3 or any convenient arrangement may be used, the following general method of procedure being recommended:

1. The soil shall be tested at a level at which it is proposed to place the bottom of the foundations of the structure.

2. The area loaded shall be not less than 2 nor more than 4 sq. ft.

3. The total load shall be determined by increasing the unit loadings proposed to be used in the design by 50 per cent.

4. The total load shall be applied and left undisturbed for at least four days; at the expiration of such time, there shall be no appreciable settlement, otherwise a lower unit loading shall be assumed and the test repeated until satisfactory.

For cheaper structures (between $1000 and $2000 total cost) a cheaper method of soil testing is to be preferred. A device applying the same principal but designed to test a smaller area will probably give results excellent for comparative purposes.

The greatest care in designing foundations may be rendered of no avail through insufficient care in the preliminary study of stream conditions and precaution against erosion.

It is recommended that after special study of the probable future behavior of the waterway, as outlined in a former paragraph, the foundations in every case be placed well below the line of possible maximum erosion.

The use of piling not only increases the bearing value of the foundation but acts as a safeguard to the foundation in time of flood where erosion not ordinarily anticipated may expose and undermine the foundation.

Fig. 4 is a 24-ft. slab bridge constructed over one of the newer streams in the Iowan Drift region. The superstructure would have carried the maxi-

mum heavy city loading recommended herein within the allowable recommended working stresses in concrete and steel. The substructure against a horizontal earth pressure equivalent to a fluid pressure of 40 lb. per cu. ft. would have developed a toe pressure of only 1.3 tons per sq. ft. In view of the fact that the foundations were of stiff clayey material such a design would appear safe for all time. But the stream was not yet to its local base level and the foundations were unprotected or, better, *uninsured* by piling, and during the heavy rains of September, 1914, the channel was scoured to a point

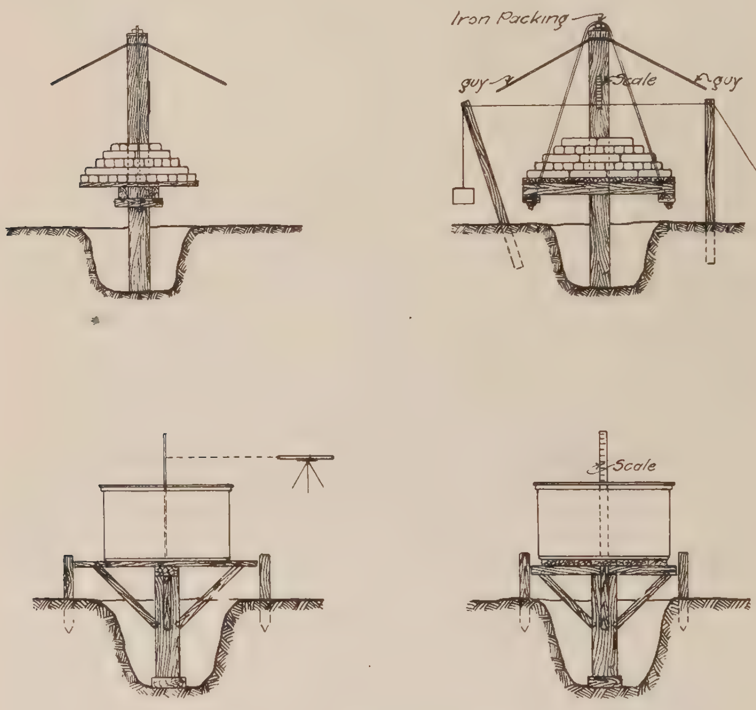

FIG. 3.—TWO TYPES OF SOIL-TESTING RIGS.

8 ft. below the former level, with the result as shown. Piling would have saved this structure until the channel was riprapped or a cofferdam built around the abutments and filled with concrete. In view of many such instances, it would appear that whenever piling can be driven at any reasonable cost, they are justified, especially if there is the remotest possibility of any erosive action.

On the other hand, it is obviously desirable to carry the substructure down a few feet, when by so doing, a hard endurable foundation material

may be encountered, rather than to drive short piles, which are not always satisfactory. It is therefore recommended:

First.—That for all except culvert work, whenever piling can be driven at any reasonable cost, they are justifiable and should be used except under conditions as noted below.

Second.—For locations where a hard, dense material is encountered at a depth below the foundation elevation such as to necessitate the use of very short piling, that the foundation be carried to this elevation rather than piled.

FIG. 4.—SHOWING COLLAPSE OF SLAB BRIDGE UNDER ERODING STREAM, DUE TO FAILURE TO INSURE FOUNDATION BY PILING.

(This structure was designed within safe unit stresses and a foundation pressure of only 1.3 tons per ft., the failure being an erosion failure entirely.)

The *Engineering News* formula is the most generally used for proportioning piling, viz:

$$B = \frac{2WH}{S+1}$$

where W = Weight of hammer in pounds,

H = Fall of hammer in feet,

S = Penetration (average of last few blows) in inches per blow,

B = Bearing power of pile in pounds.

Recent tests furnish data to indicate that the above formula may be used with the understanding that for loads causing a settlement of $\frac{1}{4}$ in. the factor of safety is two.

For concrete piles, both premolded and those cast in place, considerable experimental data have been collected but many more are needed. Your committee hopes to pursue this subject further at a later date.

The Fixed Arch.—The fixed arch is an elastic structure whose design is commonly predicated on the immovability of the foundation supports. Any failure to produce supports *absolutely rigid and fixed,* operates to throw the arch ring under greatly increased stresses.

If L represents the span length in feet and the coefficient of thermal expansion be taken as 0.000006, a spreading or horizontal displacement of the abutments of h inches will result in stresses equivalent to those induced by a temperature drop of t degrees where $t=h/0.000072L$. Thus a spreading

FIG. 5.—CONCRETE ARCH IN JACKSON COUNTY, IOWA, WHICH COLLAPSED DURING HIGH WATER DUE TO FAILURE TO PROVIDE FOUNDATION SUFFICIENT IN DEPTH AND AREA AND NEGLECT TO USE PILING.

of $\frac{1}{2}$ in. in a 100-ft. span results in stresses in the ring equivalent to a temperature drop of 70° F. much more than any arch is ordinarily designed to withstand.

For this reason special precaution should be taken in the foundation design, and the use of this type restricted to those locations where natural foundations are of the best or when the expenditure necessary to amply insure against any displacement is justified. Foundations for this type of construction should, in general, be carried much deeper and piling used wherever it is possible to drive it. In designing the foundation, unless absolute data to the contrary are available, the minimum value for safe loading of foundation given in the foregoing table should be used, and when piling are used, they should preferably be driven on a batter so that they will be, as

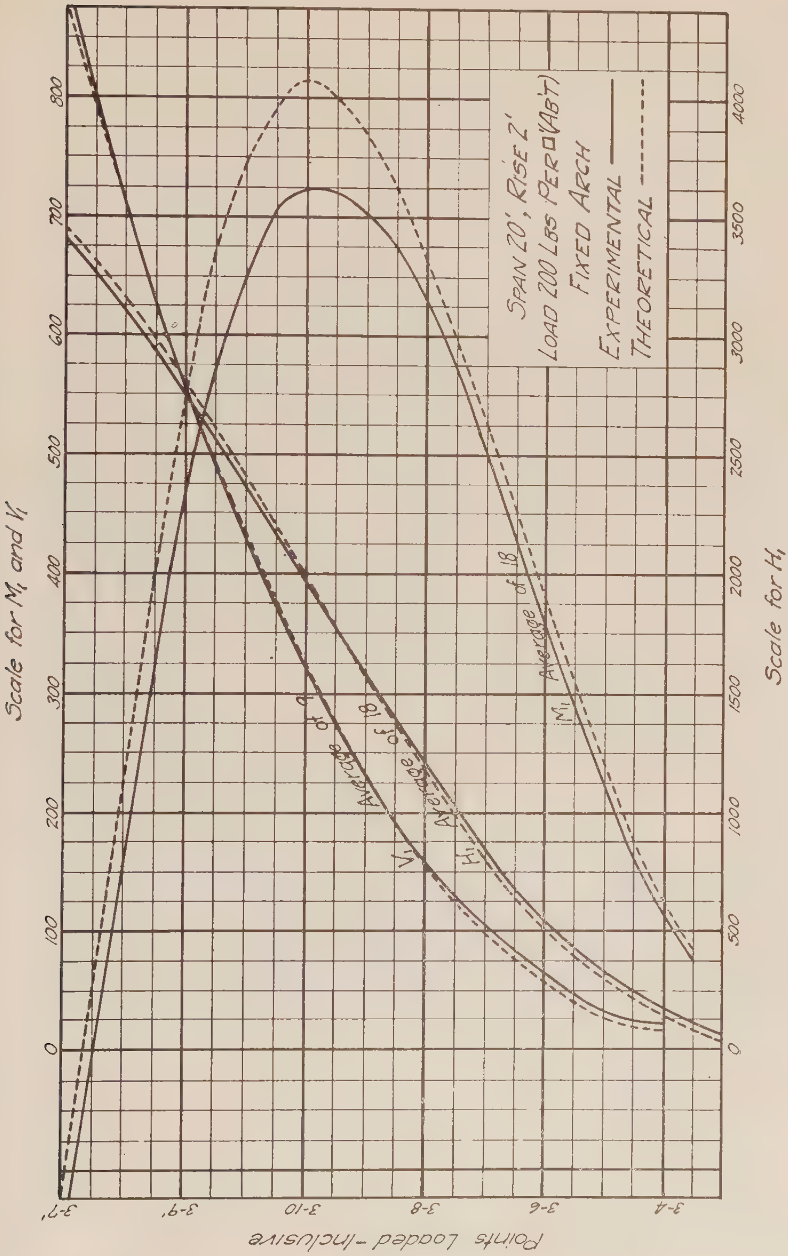
SPAN 20', RISE 2'
LOAD 200 LBS PER □'(ABT)
FIXED ARCH
EXPERIMENTAL
THEORETICAL
Scale for M_1 and V_1
0
100
200
300
400
500
600
700
800
Scale for H_1
0
500
1000
1500
2000
2500
3000
3500
4000
Points Loaded - Inclusive
3-7'
3-9'
3-10
3-8
3-6
3-4
V_1 Average of 9
H_1 Average of 18
M_1 Average of 18

FIG. 6.

nearly as practicable, parallel to the direction of the line of thrust at the base of the substructure.

Your committee wishes to submit Fig. 5 as emphasizing the disastrous results of failure to provide for proper footing for this type of construction. The view is of a 60-ft. arch span constructed in Jackson County, Iowa, the history of which is briefly as follows: Constructed, 1910; cost, $3,041; collapsed, 1912, failure being due to a spreading of the abutments during a period of exceptionally high water. The spreading in turn was the result of an undermining of foundations insufficient in both depth and area and unprotected by piling. This is one of many such spans which, during the past few years, have collapsed for similar reasons.

The stresses in the arch ring are calculated from the elastic properties of the material on the hypothesis that such material remains uncracked. This so-called elastic theory has been amply proven by experiment (notably

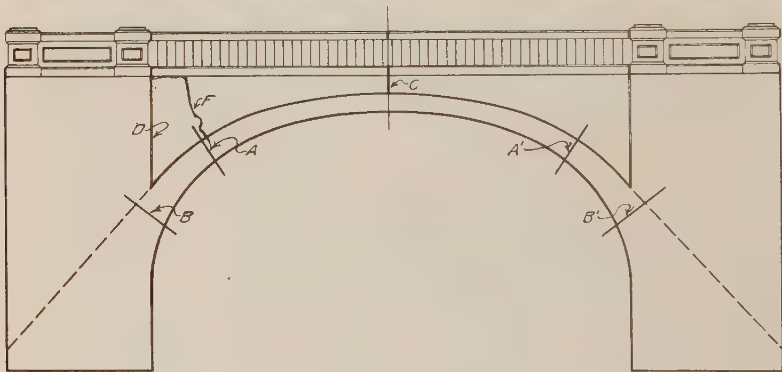

FIG. 7.

of the Austrian Society of Engineers and Architects and those of Prof. M. A. Howe) to hold for stress values under the point of rupture in the concrete.

Fig. 6, taken from the work of Prof. Howe, is inserted as illustrating the close agreement between theoretical and experimental values. (See *Railway Age Gazette*, March 26, 1909.)

When the ultimate strength of the concrete in tension is reached, the neutral axis rises and is no longer coincident with the arch axis. However, the section still possesses elastic properties and the original law of stress distribution modified by the new conditions probably still holds. Experimental data along this line are badly needed, as it is not possible to design against temperature stress within the limits of the tensile strength of concrete.

Experimental data for the Northern Middle West are such as to indicate an internal temperature range expressed by the formula $Y=90-0.53X$, where Y is the yearly internal temperature range in degrees Fahrenheit and X is the distance from the nearest exposed surface in inches. For an aver-

age thickness of arch ring of 36 in., X will equal 18, and Y, the temperature effective in producing stress in the ring, should therefore be taken as about $90-0.53\times 18$ or 80° F. For very thick rings, this value will be correspondingly reduced. The above formula applies to latitudes having an annual varia-

FIG. 8.—TYPICAL DIAGONAL CRACKS AT THE QUARTER POINT OF THE ARCH RING DUE TO A PARTIAL FIXING OF THE ENDS BY SPANDREL WALLS WITHOUT EXPANSION JOINTS.

tion in temperature of 135° F. For warmer latitudes, this value may be somewhat decreased.

The application of the above formula may (especially for highway loadings) result in stress values greater than the combined dead and live

load stresses. Many times, such stresses are sufficient at both crown and spring line to reverse the dead load and induce tension at both intrados and extrados. In the analysis of arches having no intradosal reinforcement at spring line, nor extradosal reinforcement at crown, your committee has, in many cases, found a tension under temperature and dead load varying from 100 to 200 lb. per sq. in. They have also observed many cases of cracking apparently due, at least in part, to this cause. For this reason, it seems the part of prudence to require, in general, that all arches be doubly reinforced at both crown and spring line.

The economical curve for the arch axis is doubtless very near the linear arch or equilibrium polygon for the given assumptions of dead loading. As

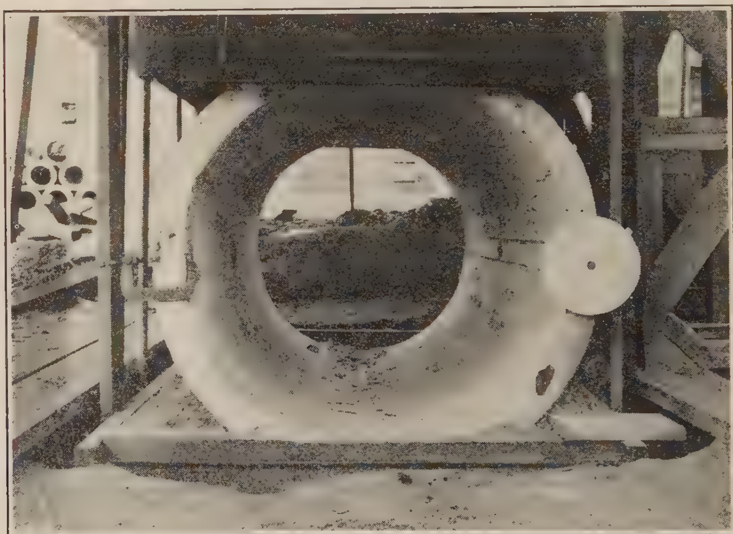

FIG. 9.—VIEW OF REINFORCED CONCRETE PIPE SHOWING FAILURE AT QUARTER POINT WHERE SINGLE LINE OF REINFORCING CROSSES NEUTRAL AXIS.

this requires a knowledge of the unit weights of filling material under the degree of compactness and drainage which will obtain in the finished construction, and also a knowledge of the amount of such load transmitted to the arch, experimental data along this line are especially needed in this connection.

The fundamental equations for vertical, horizontal and angular displacement as generally used, are derived by neglecting the distortion due to axial compression, although such compression is present and may considerably modify the stress values. Such axial compression (rib shortening) may be considered separately, its action being that of a temperature drop of $t°$ where $t = f_c/Ec$, where f_c is the average compression in the ring, E and c are

respectively the modulus of elasticity and coefficient of thermal expansion of the concrete.

Your committee recommends that all arch structures be figured for rib shortening or axial compression stresses due to the compression of the dead load thrust.

If the arch shown in Fig. 7 were to be analyzed as elastic from B to B' and the unit stresses determined, and then analyzed as elastic from A to A', it would be found that for ordinary axial curves the stresses at A would be greatly increased by the latter method of analysis. Where spandrel walls are attached to the arch ring, unless a vertical expansion joint is placed both

FIG. 10.—TYPICAL EROSION EXPOSING THE FOUNDATION OF CULVERT STRUCTURES.

(A failure to intelligently study the stream and to provide foundation against erosive action of channel.)

at the crown and at D, the latter condition may exist to some extent at least, tending to form cracks as shown at F. Much cracking of this type has been observed where no vertical expansion joint is placed at D (as shown in Fig. 8). It is recommended, therefore, that especial care be taken in the provision for expansion joints in the spandrel walls at the crown, at the springing line, and at such intermediate points as are deemed advisable.

In concluding the subject of arch design, your committee wishes to unhesitatingly condemn empirical and "rule of thumb" methods unless the resulting structures are checked by the elastic theory. While it is true that many of the variables entering the problem are not as yet evaluated by experi-

ment, yet the elastic theory applied to arch design has resulted in structures' the majority of which are as yet free from defect, while empirical methods have resulted, in many instances, in inefficiently and improperly designed construction, rapidly developing cracks and defects and operating in many communities to give concrete bridge construction in general an evil reputation.

Culvert Types.—For the smaller openings (spans up to 12 ft.) several types of construction are ordinarily used.

Pipe Culverts.—For the smaller spans up to between 3 and 4 ft., concrete pipe with concrete head walls and aprons may prove the economical type. These pipes should be reinforced, and special care taken in the selection of a

FIG. 11.—TYPICAL EROSION EXPOSING THE FOUNDATIONS OF CULVERT STRUCTURES.

(A failure to intelligently study the stream and to provide foundation against erosive action of channel.)

type of joint such as to eliminate as much as possible any seeping or leakage at the joints, thereby softening the foundation and causing the pipe to settle out of alignment. Where such a joint can not be obtained, it is recommended that the pipe be bedded in concrete at the joints. In tests of singly reinforced pipe (reinforcing crossing neutral axis at 45 deg. with the vertical) a tendency to the formation of cracks is noted at the point where such reinforcing crosses the neutral axis. (See Fig. 9.) Apparently the strength of such pipe may be much increased by doubly reinforcing.

Circular Culverts.—Cast over special portable forms, these have been used to a considerable extent. As a rule, these are more costly but probably

more permanent than the former type, for the joint is eliminated and the sections for construction reasons are more massive.

Culverts for Greater Spans.—For spans from 6 to 12 ft., both the box and the arch type are used. The former has to recommend it the fact that it is a statically determinate structure and also the fact that foundation settlement does not induce stresses in the superstructure. The arch is probably the more costly for low fills while for deep fills the reverse may be true.

All culverts should be designed not only to take the ordinary loading but to act as longitudinal girders transmitting any uneven super-load uniformly over the foundation area or as near uniformly as practicable.

The tendency in culvert construction is in general to place the foundations too high. The erosion in small streams is apt to be much greater proportionally than for the larger ones in the same area. Fig. 10 and 11 illustrate the erosion and lowering of base level possible in two years' time, the foundations being more or less completely undermined. Precaution in this regard is strongly recommended.

In concluding its preliminary report, your committee wishes to point out that its aim has not been to cover the entire field of concrete bridge construction but to point out the main problems confronting the designer, to emphasize the need of experimental data and to promote interest and discussion of the subject.

In a future report, it is hoped to take up other problems, to study other types, and to amplify the tentative recommendations in the light of results of further experiment.

C. B. McCULLOUGH, *Chairman.*

DISCUSSION.

Mr. Lovis.

Mr. Lovis.—The T-beam type of bridge is mentioned in the report, and it has been suggested that further consideration of this subject be given. A statement of the experience of the Massachusetts Highway Commission with concrete highway bridges may be interesting, especially in the use of the T-beam type for bridges of spans of less than 40 ft.

They began to build small reinforced concrete box culverts in 1902. These culverts varied in size from 2-ft. to 6-ft. spans, and the same form of construction was extended to 12-ft. openings. For greater spans than this, T-beams have generally been used, and, as experiments and experience have warranted it, the span has been extended up to 40 ft. The clear roadway width has never been less than 21 ft., and within the last few years all bridges have been designed for a width of not less than 23½ ft. between guards.

The accompanying table shows that the T-beam is more economical than the through girder for roadways of greater width than 15 ft. As for the deck girder, there have probably never been conditions which would warrant its use, as the necessary headroom has been lacking.

For spans greater than 40 ft., arches have been designed, both on account of economy and on account of the architectural possibilities of this form of construction. It is true, also, that the arch has been found an economical form for smaller openings when the headroom has been ample and the arch ring may be a semi-circle or nearly one.

As stated, the experience in Massachusetts in the construction of highway bridges resulted logically in the adoption of the types of bridges mentioned.

The following table gives a comparison of costs for T-beam and through girders for bridges of 30-ft. span and roadway widths of 10, 15, and 20 ft. The costs given are actual costs of materials and forms in place:

Comparison of the Actual Cost of Through Girder and T-Beam Types of Superstructure of 30-foot Span.

Through Girder.				T-Beam.			
	10-ft. Width.				10-ft. Width.		
Concrete........	14.52 cu. yd.	at $7.20	$104.54	Concrete.........	8.25 cu. yd.	at $11.40	$97.13
Steel...........	825 lb.	at .03	24.75	Steel............	1642 lb.	at .03	49.26
Total...........	14.52 cu. yd.	at 8.90	$129.29	Total............	8.25 cu. yd.	at 17.18	$146.39
	15-ft. Width.				15-ft. Width.		
Concrete........	27.55 cu. yd.	at $7.05	$194.23	Concrete.........	13.16 cu. yd.	at $11.30	$149.45
Steel...........	1640 lb.	at .03	49.20	Steel............	2830 lb.	at .03	84.90
Total...........	27.55 cu. yd.	at 8.84	$243.43	Total............	13.16 cu. yd.	at 17.71	$233.04
	20-ft. Width.				20-ft. Width.		
Concrete........	45.15 cu. yd.	at $7.00	$316.05	Concrete.........	19.04 cu. yd.	at $11.25	$214.20
Steel...........	2350 lb.	at .03	70.50	Steel............	4180 lb.	at .03	125.40
Total...........	45.15 cu. yd.	at 8.51	$386.55	Total............	19.04 cu. yd.	at 17.84	$339.60

REPORT OF COMMITTEE ON CONCRETE ROADS AND PAVEMENTS.

During the past year the work of the committee has been confined to a revision of the four standard specifications for roads and pavements, viz:

- No. 5. Standard Specifications for One-Course Concrete Highways.
- No. 17. Standard Specifications for One-Course Concrete Street Pavement.
- No. 18. Standard Specifications for Two-Course Concrete Street Pavement.
- No. 19. Standard Specification for One-Course Alley Concrete Pavement.

In addition, the committee submits a specification for concrete pavement between railway tracks, which conforms in substance and arrangement with the other specifications.

It is not claimed by the committee that the specifications already adopted by the Institute, with the proposed changes, are perfect, for it is recognized that minor changes would improve them slightly. On the other hand, it is not desirable to change standard specifications annually, since such changes prevent their general adoption and lead to the development of other specifications which have not received the careful thought and been tried by the test of so much actual work under them as have those of the Institute. So in suggesting the following changes the committee feels that it is making only such recommendations as are capable of being demonstrated by facts and data to be really necessary.

Standards 5, 17, 18 and 19,* clause 2, *Aggregates* first paragraph, omit the last two sentences and substitute for them, "Aggregates containing frost or lumps of frozen material shall not be used." In the second paragraph change the last sentence to read, "Fine aggregates shall not contain vegetable or other deleterious material nor more than three (3) per cent by weight of clay or loam."

After this paragraph insert the following new paragraph:

"Routine field tests shall be made on fine aggregate as delivered. If there is more than five (5) per cent of clay or loam by volume in one (1) hour's settlement after shaking in an excess of water, the material represented by the sample shall be held pending laboratory tests."

The old specifications did not indicate whether the test for clay and loam was to be made by weight or volume. The new clauses enable the field engineer to ascertain by a test easily made with a graduated glass whether the material is certainly satisfactory or should be submitted to a laboratory test, which will involve a delay that it is desirable to avoid if practicable.

* The four standards were printed in the *Journal* of the American Concrete Institute for April, 1915, p. 109.

The next paragraph of the same clause of all specifications should read as follows: "Fine aggregate shall . . . show a tensile strength . . . equal to or greater than . . . " etc. This is the substitution of the words "equal to or greater than" for "at least equal to."

Standard 18, clause 3, in the specification for No. 1 Aggregate for Wearing Course and for No. 2 Aggregate for Wearing Course, change "gravel" to "pebbles" in the first sentence.

All Standards, clause 5, *Reinforcement*, omit the second sentence.

All standards, clause 6, *Joint Filler*, add at end of paragraph, "Prior to submitting bid, the contractor shall obtain approval of the engineer for the joint filler which he proposes to use."

Standard 5, clause 10, *Engineer's Stakes*, change "grade line" to "established grade."

Standard 5, clause 11, and Standards 17, 18 and 19, clause 10, change the title of the clause from "*Excess Materials*" to "*Free Haul.*"

All Standards, clause 12, *Cuts and Fills*, first paragraph, omit the following sentence, "The engineer shall determine the depth of material to be thus compacted."

Standards 17, 18 and 19, clause 14, *Catch-basins*, substitute for the words "at his expense" the words "at the price shown under this item in his bid."

Standard 5, clause 6, and Standards 17, 18 and 19, clause 15, *Construction*, omit the first paragraph. Change the first sentence in the last paragraph to read: "When the concrete pavement is to be constructed over an old roadbed composed of gravel or macadam, the old roadbed shall be entirely loosened and the material spread for the full width of the roadbed and rolled."

Standard 5, clause 21, *Width, Thickness of Concrete and Crown*, change the note to read, "the thickness of the concrete at the edges shall be not less than six (6) inches. The crown shall be not less than one one-hundredth ($\frac{1}{100}$) nor more than one fiftieth ($\frac{1}{50}$) of the width."

Standard 17, clause 20, *Width, Thickness of Concrete and Crown*, change the note to read, "The thickness of the concrete at the sides shall be not less than six (6) inches and at the center not less than two (2) inches more than the thickness at the sides. When pavements twenty (20) feet or less in width are to be built on approximately level ground and a flat subgrade is to be used, sufficient fall for drainage at the sides of the pavement along the curb shall be provided by giving the roadbed the same grade as that proposed for the gutter. The crown shall be not less than one one-hundredth ($\frac{1}{100}$) nor more than one fiftieth ($\frac{1}{50}$) of the width."

Standard 18, clause 20, *Width, Thickness of Concrete and Crown*, change the note to read, "The thickness of the concrete base at the sides shall be not less than five (5) inches and at the center not less than two (2) inches more than the thickness of the sides. The thickness of the wearing course shall not be less than two (2) inches. When pavements twenty (20) feet or less in width are to be built on approximately level ground and a flat subgrade is to be used, sufficient fall for drainage at the sides of the pavement along the curb shall be provided by giving the roadbed the same grade as that proposed for the

gutter. The crown shall be not less than one one-hundredth ($\frac{1}{100}$) nor more than one fiftieth ($\frac{1}{50}$) of the width."

Standard 19, clause 20, *Width, Thickness of Pavement and Cross-Section* in last sentence change "pitch" to "dish."

Standards 17 and 18, clause 21, *Width and Location*, change the last sentence to read, "All joints shall extend through the entire thickness of the pavement and curb (when integral curb is specified) and shall be perpendicular to the surface of the pavement. In pavements with integral curb the joint shall be continuous in a straight line through the pavement and curb."

Standard 19, clause, 21, *Width and Location*, change second sentence to read, "When the pavement is laid adjacent to buildings or other masonry structures, a joint shall be constructed between the pavement and the structure."

Standard 5, clause 23, *Joint Filler*, omit the word "transverse."

Standard 5, clause 25, and Standards 17, 18 and 19, clause 24, *Unprotected Joints*, omit the second sentence, as the subject is covered later in the revised specifications.

Standard 5, clause 27, and Standards 17, 18 and 19, clause 26, *Mixing*, substitute the following for the present clause, "The materials shall be mixed in a batch mixer approved by the engineer, and irrespective of the size of the batch and rate of speed used, mixing shall continue after all materials are in the drum for at least one (1) minute before any part of the batch is discharged from the drum. The drum shall be completely emptied before receiving material for the succeeding batch. The drum shall revolve at a rate of speed specified for the particular mixer used by the contractor but not less than twelve (12) revolutions per minute. The volume of the mixed material used per batch shall not exceed the manufacturer's rated capacity of the drum in cubic feet of mixed material."

This recommended clause is largely a rewording of the present clause in order that it may be definitely understood that the one-minute period of mixing does not begin until all the materials are in the mixer and no materials are to be discharged from the mixer until the full minute has elapsed.

Standard 18, clause 29, *Cement Required*, change "at least" to "not less than" wherever the former occurs.

Standard 5, clause 31, and Standards 17, 18 and 19, clause 30, *Reinforcing*, change the first paragraph to read, "Concrete pavements twenty (20) feet or more in width shall be reinforced. The reinforcement shall have a weight of not less than twenty-eight (28) pounds per one hundred (100) square feet.* The ratio of effective areas of reinforcing members at right angles to each other may vary from 1:1 to 4:1. The spacing between parallel lines of reinforcing members shall not be more than eight (8) inches. A reduction of three (3) pounds from the weight specified shall be allowed for these types of reinforcement not requiring extra metal at intersections."

The changes in the requirements for reinforcement have been made after consultation with a number of engineers and representatives of manufacturers

* The committee is of the opinion that the weight of reinforcement for streets over twenty-five (25) feet wide should be greater than twenty-eight (28) pounds per one hundred (100) square feet.

of reinforcement. The present state of our knowledge of the functions actually performed by the reinforcement is unsatisfactory and until we know more definitely what requirements are necessary and what are unnecessary in order to obtain reinforcement which will serve its purpose without being unnecessarily expensive, the committee advises the adoption of the new clause, under which all types of reinforcement may be used. The committee does not feel that the available data warrant it in specifying at this time whether the greater strength in the reinforcement should be developed lengthwise or across the roadway, and accordingly leaves this decision for the present with each engineer using the specifications.

Standard 5, clause 38, Standards 17 and 19, clause 31, *Placing Concrete*, second paragraph, add the following, "If the concrete is placed in two courses, as when reinforcement is used, any dirt, sand or dust which collects on the base course shall be removed before the top course is placed." Also add the following note to this paragraph, "Note.—The concrete above the reinforcement shall be placed immediately after mixing and in no case shall more than forty-five (45) minutes elapse between the time that the concrete below the reinforcement has been mixed and the concrete above the reinforcement is placed." At the end of the last paragraph omit, "in the work."

Standard 5, clause 33, *Finishing*, first paragraph, change third sentence to read, "Any holes left by removing any material or device used in constructing the joint shall be filled immediately with concrete from the latest batch deposited on the subgrade." Add to the paragraph this sentence, "The concrete adjacent to unprotected joints shall be finished with a wood float which is divided through the center and which will permit finishing on both sides of the filler at the same time."

Standard 17, clause 32, *Finishing*, first paragraph, change second sentence to read, "Any holes left by removing any material or device used in constructing the joint shall be filled immediately with concrete from the latest batch deposited on the subgrade." At the end of the paragraph add, "The concrete adjacent to unprotected joints shall be finished with a wood float which is divided through the center and which will permit finishing on both sides of the filler at the same time. The concrete for a distance of eighteen (18) inches out from the curb line shall be finished with a steel trowel.

Standard 18, clause 32, *Placing Concrete*, third paragraph, retain the first sentence and substitute for the remainder of the paragraph the following, "If dirt, sand or dust collects on the base it shall be removed before the wearing course is applied. The reinforcing metal shall be placed upon and slightly pressed into the concrete base immediately after it is placed."

Standard 18, clause 36, *Finishing*, first paragraph, make same changes as for Standard 17, clause 32, first paragraph, stated above.

Standard 19, clause 32, *Finishing*, first paragraph, change second sentence to read, "Any holes left by removing any material or device used in constructing the joint shall be filled immediately with concrete from the latest batch deposited on the subgrade." Add the following sentence to the paragraph, "The concrete adjacent to unprotected joints shall be finished with

wood float which is divided through the center and which will permit finishing on both sides of the filler at the same time."

Standard 5, clause 34, Standards 17 and 19, clause 33, and Standard 18, clause 33, *Curing and Protection*, change third paragraph to read, "When the average temperature is below fifty (50) degrees F., sprinkling and covering of the pavement may be omitted at the discretion of the engineer."

Standard 5, clause 35, Standards 17 and 19, clause 34, and Standard 18, clause 39, change the title of the clause to "*Cold Weather Work.*"

PROPOSED STANDARD SPECIFICATIONS FOR CONCRETE PAVEMENT BETWEEN STREET CAR TRACKS.

I. Materials.

1. *Cement.*—Same as in Standards 5, 17, 18 and 19.

2. *Aggregates.*—Same as in Standards 5, 17, 18 and 19.

3. *Joint Filler.*—The filler for joints shall consist of prepared strips of fiber matrix and bitumen, or similar material of approved quality one-quarter ($\frac{1}{4}$) inch in thickness. The width of the joint filler shall be not less than one-half ($\frac{1}{2}$) inch greater than the thickness of the pavement at any point. Prior to submitting bid, the contractor shall obtain approval of the engineer for the joint filler which he proposes to use.

4. *Bitumen.*—Pitch or asphalt used for breaking of bond between sections of concrete as specified shall be of such a character as to adhere firmly to the surface of the concrete base and remain in position when solidified.

II. Preparation of Surface of Concrete Base.

5. *Placing Bitumen.*—After the concrete base has hardened the entire area upon which the concrete pavement is to be placed shall be completely covered and brought to an even surface by spreading thereon a mat of bitumen and sand.

The bitumen shall be heated in a kettle which shall insure a uniform heat throughout its contents.

As the bitumen is needed it shall be carried directly to the work and deposited evenly over the surface of the concrete base. Each square yard of surface shall be covered with not less than one-third ($\frac{1}{3}$) gallon and not more than one-half ($\frac{1}{2}$) gallon of bitumen.

6. *Placing Sand.*—Immediately after the bitumen is deposited it shall be completely covered with sand.

After the bitumen has solidified the surplus sand shall be broomed from the surface.

III Preparation of Surface of Concrete Base.

7. *Placing Bitumen.*—The surface of the concrete base over the entire area to be covered with concrete pavement shall be completely covered and brought to an even surface by spreading thereon a mat of bitumen and sand.

The bitumen shall be heated to approximately 350° F. in a suitable kettle which will insure a uniform heat throughout its contents.

As the bitumen is needed it shall be carried directly to the work in suitable size sprinkling cans and deposited evenly over the area at a temperature of not less than 250° F. Each square yard of surface shall be covered with not less than one-third ($\frac{1}{3}$) gallon and not more than one-half ($\frac{1}{2}$) gallon of bitumen.

8. *Placing Sand.*—Immediately after the bitumen is deposited it shall be completely covered with sand.

After the bitumen has solidified the surplus sand shall be broomed from the surface.

Care shall be exercised so that no bitumen shall come in contact with the rails above the rail base except as hereinafter specified.

9. *Old Track.*—Where an old track base is to be prepared for concrete pavement, it may be necessary to use more bitumen and sand to secure a smooth, even surface. Any depressions over one (1) in. in depth shall be filled with mortar or concrete.

IV. FORMS.

10. *Materials.*—Same as clause 17 in Standards 17, 18 and 19.
11. *Setting.*—Same as clause 18 in Standards 17, 18 and 19.
12. *Treatment.*—Same as clause 19 in Standards 17, 18 and 19.

V. PAVEMENT SECTION.

13. *In Track.*—The concrete shall extend from web to web of rail and have a minimum thickness of not less than six (6) inches at the center of the track.

14. *Grooved Rail.*—The concrete surface at rail shall be level with the lip of the rail and between rails conform to a straight edge placed from lip to lip of rail.

15. *T-Rail.*—The concrete surface at rail shall be one and one-eighth ($1\frac{1}{8}$) inch below top of rails.

At the center line of track the surface of the concrete shall be one-quarter ($\frac{1}{4}$) inch below the top of the rails. The pavement shall have a parabolic curve passing through these three points located as above and in line at right angles to the center line of the track.

16. *Between Tracks.*—The center shall extend from web to web of rails and have a minimum thickness of not less than six (6) inches.

The surface of the concrete shall be one-quarter ($\frac{1}{4}$) inch below the top of T-rails and three-eighths ($\frac{3}{8}$) inch below the top of grooved rails at rails and crowned at right angles to center line of tracks in an arc of a circle passing through a point on the center line between tracks one-half ($\frac{1}{2}$) inch above the top of rail.

VI. JOINTS.

17. *Location of Joints.*—When necessary to continue traffic over one track of double-track construction, a continuous joint shall be constructed on the center line between tracks.

When the concrete pavement cannot be laid at the same time that the pavement is being placed on the car tracks and it is necessary to operate over the tracks while the street pavement is being laid, then the street and car-track pavements shall be completely separated by a one-quarter ($\frac{1}{4}$) inch longitudinal joint which shall be placed at the outside edge of the concrete track base. When the concrete pavement is laid at the same time the track is paved, the street pavement should be continuous without longitudinal

joints to be outside of the rail. The bond between the rail and concrete at this point, however, shall be broken by painting the outside of the rail with bitumen.

18. *Joint Filler.*—All longitudinal joints shall be formed by inserting during construction and leaving in place the required thickness of prepared strips of fiber matrix and bitumen or similar material of approved quality, which shall extend through the entire thickness of the pavement.

19. *Longitudinal Joints.*—All longitudinal joints shall extend through the entire thickness of the pavement and the filler shall project not less than one-half (½) inch above the finished surface. Before the pavement is open to traffic, the joint filler shall be cut off to a height of one-quarter (¼) inch above the surface of the pavement.

20. *Construction Joints.*—When necessary to discontinue the placing of concrete for a period of more than thirty (30) minutes, the concrete shall be brought to a straight perpendicular edge at right angles to the center line of the track by means of a bulkhead form.

When construction is resumed the form shall be removed and the concrete placed flush against the section formerly placed.

VII. MEASURING MATERIALS AND MIXING CONCRETE.

The six clauses in this section are the same as the clauses in section VIII of Standards 5, 17 and 19.

VIII. PLACING CONCRETE.

27. *Placing Concrete.*—After mixing the concrete shall be deposited rapidly in successive batches upon the finished bituminous and sand cushion to the required depth and for the entire width of the section to be concreted in a continuous operation. The concrete shall be thoroughly tamped until surplus mortar is brought to the surface. A foundryman's peen or similar tool shall be used to compact the concrete thoroughly against the rails, rail fastenings and tie rods.

28. *Finishing.*—The surface of the concrete shall be struck off by means of a template or strike-board which shall be moved with a combined longitudinal and crosswise motion. After being brought to the established grade with a template or strike-board the concrete shall be finished from a suitable bridge, no part of which shall come in contact with the concrete. The concrete shall be finished with a wooden hand float in a manner to thoroughly compact it and produce a surface free from depressions or inequalities of any kind. The concrete adjacent to longitudinal joints where a joint filler is required shall be finished with a wood float which is divided through the center and which will permit finishing of the car track pavement at the same elevation as the adjoining street pavement.

The finished surface of the pavement shall vary not more than one-quarter (¼) inch from the true shape when measured by a template resting upon the top of the rails and at right angles to the center line of the track. The surface of the pavement for a distance of six (6) inches each side of the rails shall be

finished to dimensions exactly as hereinbefore specified; no variations either way will be permitted.

After the concrete has been struck off, workmen shall not be permitted upon it until thoroughly hardened.

IX. PROTECTION.

The two clauses in this section are the same as clauses in section XI of Standards 5 and 19 and XII of Standards 17 and 18.

It will be observed that even in the special sections of the proposed Standard Specifications for Concrete Pavement between Street Car Tracks as much as possible of the wording of the Standards for other concrete pavements has been retained.

A. N. JOHNSON, *Chairman.*

DISCUSSION.

Mr. Tashycan.

Mr. Tashycan.—The requirements for protected joints are apparently too specific, barring out some types which may have advantages under some conditions. General specifications should not debar a material or method of construction which is definitely useful for some purposes.

Mr. Kinney.

Mr. Kinney.—The requirements for protected joint metal specify merely the minimum depth and thickness which the committee believes will give satisfactory service. Lighter joint plates can hardly be considered to afford proper protection. The criticism can hardly be considered important for every type of joint plate on the market of which I have knowledge complies with the requirements.

Mr. Collings.

Mr. Collings.—The proposed specifications apparently pay no attention to the elastic limit of the steel used for reinforcement and 28 lb. of metal are required whether it be mild steel of 35,000 lb. elastic limit or wire of 60,000 to 70,000 lb. elastic limit. In other engineering work allowance is made for this difference in quality, and it is not evident why no such allowance is made in the reinforcement for concrete pavements.

Mr. Johnson.

Mr. Johnson.—The present specifications contain a clause fixing the ultimate strength of the steel which the committee recommends should be omitted. Until our knowledge of what reinforcement actually accomplishes in a concrete pavement is more precise and comprehensive than it is now, the committee holds that it is best to permit the use of any grade of steel. The committee accordingly conferred with many persons who might help it to gain a better comprehension of the subject of reinforcement, and with this assistance and advice it prepared the revised requirements submitted in its report which permit the use of any commercial type of reinforcing material. Our information derived from experience shows that in pavements over 20 ft. wide there are not so many cracks where reinforcement is used as there are where it is omitted, and we have accordingly recommended its use. The scientific authorities we consulted have advised us to base our recommendations on the teachings of experience. Unfortunately it is difficult to find pavements exactly alike in all construction and service details except that some have steel and others do not. Without such pavements to study our information is very general and progress to greater detailed accuracy must be quite gradual.

Mr. Pease.

Mr. Pease.—There are something like 7,000,000 to 8,000,000 sq. yds. of reinforced concrete pavements in the country. It is rather strange that we cannot reach some conclusion from this yardage as to the value of reinforcement and how much of it should be used. The subject is certainly important enought to merit a special investigation.

President Wason.

President Wason.—If there is no objection the report will be printed and the committee continued. (No objection was made.) As the report was not distributed to the members thirty days before the Convention it cannot be formally adopted this year.

THE CONSTRUCTION OF THE TORONTO TO HAMILTON HIGHWAY BY DAY LABOR.

By H. S. Van Scoyoc.*

Provincial aid has been given to highway construction in Ontario for more than twenty years, an act having been passed establishing a voluntary county road system and providing for the payment of a fixed proportion of the construction costs by the province. More than half of the counties have adopted the plan and satisfactory results have been secured under it. Present-day conditions seemed to call for some modifications in this scheme, however, and in 1913 a Special Public Roads and Highways Commission was appointed which made its report in 1914. As a result of this report a new bill known as The Ontario Highways Act, 1915, was assented to April 8, 1915, but did not come into force until January of this year. It provides in a very comprehensive manner for both construction and maintenance work and states the basis upon which provincial grants are given. The Toronto to Hamilton Highway is being built under a special act, although it practically conforms to the new highways bill. It was undertaken previous to the going into effect of the new act on account of unusual conditions.

With the entry of the British Empire into the great war in August, 1914, industrial conditions in Canada became very much unsettled. Thousands of men were out of employment and, especially in the larger centers of population, the problem of preventing suffering during the approaching winter was a serious one.

In a group discussing this situation late in August were Messrs. George H. Gooderham, G. Frank Beer and Mark H. Irish, all members of the Provincial Parliament of Ontario from the city of Toronto. The advisability of highway construction as a relief measure was suggested. Attention was drawn to efforts already made to provide a concrete highway between Toronto and Hamilton. It was hoped that special prices could be secured on stone and cement, as the producers were anxious to keep their plants in operation. Practically all of the municipalities interested had expressed their approval of the construction of the highway. A statement of the probable cost of the undertaking was prepared, together with a plan for financing it. A petition was presented to the Minister of Public Works setting forth the facts as they appeared. Reports were made on the petition by the Deputy Minister of Works and the Engineer of Highways. A commission was appointed by an Order-in-Council dated September 17, 1914, Parliament not being in session. By the end of October the necessary resolutions had been agreed to by the interested municipalities. On November 4, 1914, the commission appointed a chief engineer and on November 6, 1914,

* Associate Member Canadian Society of Civil Engineers; Associate Member American Society Civil Engineers; Chief Engineer, The Toronto and Hamilton Highway Commission.

construction work was begun. Preliminary surveys had been made by the Provincial Engineer of Highways, Mr. W. A. McLean, and they were used as the basis for the early work.

By the original plan of financing, definite sums were contributed by Toronto and Hamilton, a stated contribution per mile was made by the provincial government and by the municipalities through which the highway passes. The balance of cost was to be provided by a local improvement tax; the manner in which it was to be levied being left with the various municipalities. The commission, on the guarantee of the province, issued debentures for a period of five years, at which time they will be replaced by debentures issued by the various municipalities. The municipal debentures are to be issued for a term not to exceed twenty years. Maintenance is provided for until 1939.

Until the Toronto and Hamilton Highway Act was in effect, funds for carrying on the work were secured on the personal guarantee of Mr. George H. Gooderham, the chairman of the commission.

As the work was undertaken to provide relief and as the standard for employment was need rather than fitness for the class of work, no feasible method could be devised for doing the work by contract. It was accordingly undertaken by day labor. The greatest need was in the two large cities, Toronto and Hamilton, and as quickly as possible work was begun near them. The number of men for whom work had to be provided was so large that one gang was succeeded by another after a limited period of employment, so that there was constant changing. Many of the men were clerks and mechanics unused to construction work. Many of them were thinly clad and some of them insufficiently fed. Almost every day was pay day and in addition the foremen were provided with funds from which gloves and other necessary articles of clothing could be purchased. As the distance from street railway lines increased is was found necessary to provide camps. Work was continued through the winter, dynamite being used to remove the frost and teams plowed day and night to keep the materials loose.

Under these conditions no records were broken as far as low excavation costs are concerned. The commission did not find a way to employ relief labor without adding to the cost of the work. It did furnish employment to men with families willing to work but unable to find anything else to do. Fortunately, this state of affairs no longer exists. No one in Canada desiring to work is at present unemployed.

General Specifications.

The highway has a minimum width of 66 ft., the commission controlling an additional strip 20 ft. wide on either side as far as the erection of fences or buildings is concerned. The roadway is graded to a level cross-section having a minimum width of 26 ft. This width has been increased in cuts and in the larger fills. The maximum grade is 4 per cent and the minimum radius on curves 300 ft. Bridges and culverts have been widened or replaced, the minimum clear width being 26 ft. This width has been increased in or near

villages and towns to provide for sidewalks. All bridges and culverts so far constructed have been of plain or reinforced concrete. Practically all pipes used under farm approaches or under the highway are of reinforced concrete. The standard width of concrete on the highway is 18 ft., the depth at the edges being 6 in. and at the center $8\frac{1}{4}$ in. The crown is parabolic. In some of the towns the width has been increased, the business section in Oakville, for example, being paved to a width of 50 ft. and a portion of the residential section to a width of 30 ft. In addition to the main highway, which is approximately 36 miles in length, a secondary road 7200 ft. long has been paved with concrete for a width of 9 ft. In general the concrete is not reinforced. In all cases where the width is greater than 18 ft., reinforcement has been used, however, as well as where foundation conditions seemed to require it.

The specifications agree very closely with the standard specifications of the American Concrete Institute for one-course concrete roadways. The proportions, however, are $1 : 1\frac{1}{2} : 3$, and up to the present time no difficulty has been experienced in securing sufficient mortar for finishing the surface.

INVESTIGATION OF MATERIALS.

In a preliminary investigation fine aggregate from 38 different sources was subjected to a granulometric analysis, a determination of the loam content and a comparison of the tensile strength of 1 : 3 briquettes with briquettes made from standard Ottawa sand. A superficial examination of the mineralogical composition was also made. These tests showed that it was possible to secure a fine aggregate grading satisfactorily, with a low loam content the mortar having a tensile strength at least equal to that obtained with standard Ottawa sand. They showed conclusively, however, that none of the sands contained less than fifty per cent of limestone and they also eliminated the sands adjacent to the highway, with the exception of one beach deposit about $2\frac{1}{2}$ miles from the western end of the road. Prices were invited from the sources felt to be satisfactory, 50-lb. samples in each case accompanying the tender. More complete tests were made on the samples submitted with the intention of incorporating the results in the contracts with the successful tenders as a standard by which the sand supplied should be judged. However, with one exception the supply men refused to agree to such an arrangement. To them sand was sand to be taken or left. It was found possible to buy subject to acceptance or rejection at the pit, however, and before the season had advanced very far it was possible to incorporate a very rigid sand specification in the agreement if an inspector was maintained at the pit. Practically every car of sand that was shipped to the commission was tested for loam content and a granulometric analysis made before the car left the pit. In an attempt to get representative samples a trough was made of sufficient length to extend under the various screens, and samples weighing about 25 lb. each were taken at least four times while each car was being loaded. These samples were combined and then quartered until the proper quantities for each test were screened. In addition more

complete tests were made in the laboratory from time to time. At one pit 19 per cent of the cars loaded were rejected, but as the inspectors sampled various sections of the pits and the shippers became more familiar with what was desired, the number of rejections were greatly reduced.

COARSE AGGREGATE.

Crushed stone samples were submitted from eleven different quarries, six being limestone, four dolamite, one granite and one trap. The specific gravity absorption per cent of wear (Deval abrasion machine) and toughness (Page impact machine) were determined in each case. Stone from five of the quarries was considered satisfactory as to quality and tenders were invited, each to be accompanied by a sample weighing at least 50 lb. Tests of these were to serve as the basis by which future shipments were to be judged. In comparing prices, the specific gravity was considered. It varied from 2.68 to 3.06, or almost 15 per cent, and as the stone was bought by weight and used by volume, this was worth thinking about. An inspector was stationed at the quarry who reported daily as to the grading. In addition, samples were sent frequently to the laboratory for complete tests.

The sand and stone inspectors were also instructed to see that only cars of suitable capacity and class were loaded and to aid in having them sent where they were most needed.

CEMENT.

Arrangements were made by which the season's requirements were to be shipped from one storehouse. This storehouse was thoroughly sampled before shipments were begun and complete tests made. In addition, cars were sampled from time to time throughout the season.

SPECIAL TESTS.

During the process of the work compression cubes were taken from batches of concrete as they were being deposited. They were kept on the road for 19 days and subjected to the same curing as the concrete in the road received. They were then taken to the laboratory and crushed at 21 days, 24 days, 3 months, etc. At the same time a set was made up in the laboratory, using the same materials, but subjected to the usual treatment for test cubes. An unfortunate breakdown of the machine is interfering with these tests during the early periods, but results will be available for the three months period and later ones.

CONSTRUCTION METHODS.

In grading, customary methods were followed. Old macadam was torn up by means of scarifiers drawn by gasoline tractors. The material was thrown to one side or leveled by means of horse-drawn graders. It was kept loose by the use of pick plows and rooters. The earth was moved by slushers, wheelers or dump wagons as the distance required. Most of the ditching was done by hand, although slushers were used to some extent. About five

miles of trenching for tile drains was done by a ditching machine. The economy in the use of the ditcher came not only from a reduction in the cost of excavation, but more largely from the saving in the quantity of gravel required for backfilling. Where suitable material is not conveniently located, the advantage of a trench 10 in. wide over one excavated by hand is very evident.

The trimming of cuts and side ditches (slopes 1½ to 1) was entirely by hand.

Fine grading and the final rolling of the sub-grade were done just previous to the depositing of the materials for the concrete.

THE TRANSPORTATION PLANT.

To construct a concrete road 18 ft. wide, 6 in. deep at the edges and 8¼ in. at the center requires practically one ton of material for each running foot of roadway. If the average haul is three miles or more, as it frequently is, the handling and transporting of this material is the most serious problem in connection with the work. As an example in our particular case, more than 90 per cent of the cost of the sand as it went into the concrete was made up of handling and transportation charges. With 36 miles of road to build, the first plan was to secure three yards that would have divided the haul about evenly, making it average a little more than three miles. Unloading equipment would have been provided for two yards and one of these outfits moved to the third yard. Unforseen difficulties prevented this, however, and delays in getting the yards in working order made more yards desirable. Materials have been unloaded at six different points. To transport the material an industrial railway outfit was purchased consisting of about twenty miles of sectional track, 24 in. gage, three 7½-ton steam locomotives and 110 1½-yard V-shaped dump cars. In three of the six yards the sand and stone were unloaded by locomotive cranes and hauled by the dinkeys; in one yard the material was unloaded by hand and hauled by teams in dump wagons; in another it was unloaded by hand into dump cars and hauled by teams on the industrial track; in the other it was unloaded by hand into the dump cars and hauled by a dinkey. Four of the yards were operated day and night. The material was dumped on the sub-grade in two rows—the final dump of stone being with the track on top of the previous dump of stone. Much of the cement was hauled with the track in this position. Where possible, hauling material past a mixing outfit was avoided, but for a limited period in one instance three machines were worked from one base. To avoid unloading direct from the railway cars into the small dump cars, portable hoppers have been built on flat cars, each hopper having a capacity slightly greater than one industrial train load. There are two hoppers with each crane, one for sand and one for stone. Each has two chutes, so that two cars are loaded at one time. The hoppers have more than cut in half the time required for loading. They prevent waste in the yards and enable the cars to be loaded evenly, preventing waste on the road. They are particularly of value when the number of dump cars is limited.

MIXING AND PLACING THE CONCRETE.

The sand and stone were loaded into the skip by means of wheelbarrows. Cube mixers with booms and overturning buckets were used, five of them were ½-yd. capacity and one of them ⅓-yd. Batches remained in the drum for at least 45 seconds, the drum rotating sixteen times per minute. The concrete was leveled in the usual way and finished with wooden floats, the men working from bridges. Special attention was directed to the joints. A split tamp was first used to compact the concrete and a divided float to finish it. Joints were of bituminated felt placed every 35 ft.

Water was secured from Lake Ontario: triplex pumps driven by 8 horse-power gasoline engines forcing it through 2-in pipes.

The concrete was covered by tarpaulins as soon as floated. The tarpaulins were removed the following morning and the concrete covered with at least 2 in. of earth. The earth was kept damp for 10 days and the road closed to traffic for at least 21 days.

At the present time practically all of the bridges and culverts have been extended or replaced, about 30 miles of the grading completed and 17 miles of concrete highway opened to traffic. The early summer should see the work completed if no unforeseen difficulties occur. This section is possibly the most important link in the provincial highway that will join Windsor with Ottawa and will connect with the through highway to Montreal.

The Toronto and Hamilton Highway Commission consists of Mr. George H. Gooderham, chairman; Mr. G. Frank Beer, honorary secretary; Mr. Reuben H. Lush, Mr. W. S. Davis, Mr. M. C. Smith, Mr. T. W. Jutten and Mr. Hugh Bertram.

The division engineer is Mr. Charles Johnston and the superintendent of construction Mr. R. T. Bell.

THE COLEMAN DU PONT ROAD, DELAWARE.

By Charles Upham.*

During 1915 work was commenced on the southerly 20-mile section of the Coleman du Pont Road. As fast as the right of way is obtained, the road will be extended from the Maryland line through central Delaware toward the Pennsylvania boundary. Approximately 55 per cent of the present construction was completed before cold weather, and it is estimated that, with the six mixers now ready for work, the remainder of this section can be completed in $1\frac{1}{2}$ months.

The alignment is practically straight, averaging only one curve in every four miles, and the curves are of large radii. The longest distance between curves is about twelve miles; the shortest distance is a little over one mile.

In general the grades are slight, there being only one grade that exceeds 1 per cent on the section now under construction.

Drainage and Grading.—The flat country made special attention to drainage necessary. The location and elevation of all ditches and streams that crossed the road or were near enough to serve as a part of the drainage system, were measured. The area of this survey generally extended one-half mile on each side of the road, but in a few instances where the country was exceedingly flat, elevations were taken in ditches three-fourths of a mile from the road. The drainage principle followed was to get the water off and away from the road as soon as possible.

Generally speaking, the soil along the du Pont road is sand or clay or a mixture of these, but in some instances the road traverses a black muck that became very hard in dry weather and very soft and slimy when wet. This black soil was capable of absorbing and retaining a large quantity of water, and for this reason the greater part of it was replaced with a more suitable material.

In general, the swamps were drained and stumps removed before the fill was deposited. In one instance, however, if this was done the cost would have been excessive, so the fill was made directly on the surface of the swamp, which was floating on a mass of soft muck underlaid with a solid foundation of sand and bog ore. It was thought that the weight of the fill, which was approximately 5 ft. deep, would be sufficient to settle it to a firm foundation. This was found to be otherwise, until holes were cut through the surface of the swamp near the toe of the fill to allow the weight of the fill to force the muck up through the holes. Approximately \$1500 was saved by this operation. The fills were made in 8-in. layers, each layer being rolled before the next was placed. If the material was a drifting sand, sufficient clay was added to compact it and keep it in place.

The grading in practically all cases was light. In general, the sub-grade was left about 2 in. high and was rolled thoroughly with a roller weighing

* Chief Engineer, Coleman du Pont Road, Incorporated.

not less than six tons. Often it appeared that the roller would break up the sub-grade rather than compact it, but after this loose material was shoveled off, it left a smooth, compact sub-grade, showing the roller to have broken up the surface of the sub-grade, but to have compacted it underneath.

Concrete Slab.—The road surface on the section now under construction consists of one mile of water-bound macadam, one mile of bituminous pavement and 18.4 miles of plain and reinforced concrete.

The concrete pavement was constructed 14 ft. wide, 7 in. deep in the center and 5 in. deep on the sides, on a flat sub-grade. The proportions of the ingredients used in the concrete were one of cement, two of sand and four of stone. Hydrated lime was used in various quantities, according to the different experiments that were made. Each batch of concrete was mixed for $1\frac{1}{2}$ minutes. The sub-grade was wet before the concrete was deposited, and care was taken that all batches were thoroughly spaded together so that a homogeneous mass might result.

No standard length of slab was adopted, as it was decided that in order to learn what the maximum length of slab could be, joints should be placed only when the concrete mixer stopped for more than 15 minutes. This resulted in some very long slabs, the longest being 460 ft. Transverse cracks have appeared in the longest slabs, and because they occurred at regular intervals where the sub-grades, materials and construction were especially good, it would seem that practically all of them were due to contraction and followed a regular principle. This will assist in determining the maximum length of slab possible to construct without transverse cracks, to meet the future requirements of the du Pont road. Armor protection plates were used at each joint.

Reinforcement.—The greater part of the pavement is reinforced, the steel being 2 in. from the surface. The reinforcement did not prevent the transverse cracks entirely, but probably reduced their number. No longitudinal cracks have appeared.

While it is intended to use five different kinds of reinforcement on this section, only three have been used up to the present time. In deciding on the different reinforcements to use, it was the intention to keep the cost and the weight per square yard as near identical as possible. One part of the road is reinforced for the entire width with No. 29 American steel and wire mesh reinforcement, giving 2.52 lb. of steel per sq. yd. of pavement. Wherever the slab is less than 178 ft. long, no cracks have yet appeared. Another part of the road is reinforced for the entire width with No. 25 Kahn expanded metal, giving 2.25 lb. of steel per sq. yd. of pavement. Wherever the slab is less than 164 ft. long, no cracks have yet appeared.

Another part of the road was reinforced with No. 25 Kahn reinforcement for a distance of 4 ft. each side of the center line. More cracks appeared in this section than where the reinforcement extended the entire width of the pavement. Another section was reinforced with the same material for a distance of 3 ft. on each side of the center line. With the exception on one slab, more cracks have appeared in this section than in either of the other reinforced sections. The section where no reinforcement was used has devel-

oped cracks, but up to the present time not as many as the sections that were reinforced with the 6-ft. and 8-ft. reinforcement. One reason for this may be that this unreinforced section has not been laid as long as the others. Lighter reinforcement of the same type has been used, but these slabs have not been inspected recently. With one exception, it appears that as the percentage of reinforcement increases, the transverse cracks become fewer. The increased cost due to reinforcement was 9 cents per sq. yd. If the reinforcement was not used and the money put into concrete it would add but ½ in. to the thickness of the road.

Finishing Concrete.—On one of the sections a power templet was used to finish the concrete, which was left smooth and with an even surface. It was claimed that this machine compressed the concrete, but its use did not reduce the number of transverse cracks. On the other sections the templets were made from 4 x 4 x ½-in. angle iron, bent to conform to the crown of the road. After the laborers learned how to use these, they worked very satisfactorily. If it could be avoided, the templet was not drawn over the pavement more than once, the finishing being done with hand floats made of wood or cork, care being taken not to disturb the concrete after it was in place. This precaution resulted in there being very little matrix or mortar above the stone.

Immediately after the concrete was finished, wooden frames covered with canvas were placed over the concrete to protect it from the sun and wind. These were kept on the concrete for about 24 hours, after which the concrete was covered with 2 in. of earth which was kept wet for 14 days. After 30 days this covering was removed and the concrete allowed to harden for two weeks before traffic was allowed on it.

Materials.—Great stress was laid upon securing satisfactory materials for the concrete. After considering various aggregates, the most economical was found to be crushed trap rock. It was required to have a French coefficient of wear of at least 17 and to be the run of the crusher passing a 1½-in. ring and retained on a ¼-in. ring. Tests of the stone used showed the average French coefficient of wear to be 17.7 and the voids to be 45 per cent. The stone was shipped over 100 miles from Pennsylvania.

There is much sand in the surrounding country, but it was decided, after many tests, that none of it was suitable for concrete. The sand finally used was a dredged river sand of exceptionally good quality. This was loaded on cars and shipped about 90 miles. The grading of this sand was especially good; the large particles being nearly ¼ in. in size. By using this sand with the trap rock aggregate, we secured an exceptionally good grading of the concrete ingredients from 1½-in. pieces of stone to the fine particles of sand.

All the cement came from the Lehigh Valley district and passed the standard requirements of the American Society for Testing Materials.

With but two exceptions the water used in the concrete construction was obtained from driven wells; in one instance it was obtained from a mill pond and in another from a small stream. The tests showed this water to be as good as the well water.

All the materials were tested and inspected by the Bureau of Inspection, a department of the Coleman du Pont road that for the past three years has

been given over to testing materials, experiment and research in highway methods and materials. The stone was inspected at the quarry. The sand was sampled as it was loaded on the cars, the tests being completed while it was in transit. The cement was tested and inspected at the mill. All water that was used in the concrete was tested each week.

Besides these various tests, a testing engineer made sieve analyses of the stone and tested the sand for voids, making any slight changes in the proportions of the ingredients found necessary. The testing engineer also sent samples of stone, sand and cement to the laboratory of the Bureau of Inspection; these were taken mostly to determine just what increment of strength was obtained by increased mixing of the ingredients.

Hydrated Lime.—A number of experiments were performed with different amounts of hydrated lime up to 10 per cent by volume. The lime was placed in the hopper of the mixer with the other ingredients. While sufficient time has not elapsed to warrant any definite conclusions being drawn, the results promise to be interesting. The only conclusion that can be drawn now as to the value of lime is from the number of transverse cracks that have developed. On one section where no lime was used, the longest reinforced slab that has not developed a crack is 191 ft. When 10 per cent of lime by volume was used, the longest slab to develop no cracks was 210 ft. In another instance a slab without reinforcement and using no lime has developed two cracks, breaking the slab up into three sections of 209, 118 and 126 ft. In two slabs where 10 per cent lime was used, no cracks have yet developed; the lengths of these slabs are 336 and 210 ft.

On another section the longest slab without lime showing no cracks is 187 ft. When 5 per cent lime was used the longest slab without a crack is 245 ft. While these are not sufficient data from which any definite conclusions can be drawn, they do point to some value being obtained from the use of lime. Much more experimenting will be done along this line and at the present time the Bureau of Inspection is carrying on quite extensive laboratory tests with cement mortars, with and without the addition of lime.

The work was very carefully inspected by men who had had considerable construction experience. Previous concrete road experience was taken into consideration and the requirements of the specifications were practically standard, with the exception of the time of mixing and a few other little details. The contractors were obliged to employ bottomless measuring boxes in order that they might use just the required quantities of stone and sand in each batch of concrete. It was difficult to secure exactly the same consistency in all batches, partly on account of the varying amounts of moisture in the aggregates and partly from the character of the water control on the mixers.

THE CONSTRUCTION OF THE EASTON-ALLENTOWN ROAD.

By John T. Gephart, Jr.*

During the 1911 Session of Legislature of the Commonwealth of Pennsylvania, a law was enacted creating a system of State Highways, each being designated by a given number. One of these routes, State Highway Route No. 159 extends from the city of Allentown, in Lehigh County, passing through the city of Bethlehem, in Lehigh and Northampton counties, to the city of Easton, in Northampton County, and is one of the heaviest traveled roads in that part of our commonwealth, and connecting three of the most important eastern cities.

In the early part of the year 1915, some of the progressive and influential residents of the above cities, advocates of good roads, conceived the idea of connecting these cities with an improved road, and volunteered to the State Highway Department to furnish the stone and cement necessary for the construction of same, providing the road be constructed as a "One Course Concrete Road." This being agreed to by all parties concerned, the survey and plans were ordered made, and the preliminary work was started the latter part of June, 1915.

The Roman roads were built of large blocks of stone set in "pozzuolana," a volcanic cement, and in the construction of the Easton-Allentown road we have returned to the basic principle of the "Roman Type," but instead of quarrying blocks of stone and transporting them to the road, we have crushed the stone at the quarry and upon bringing it to the road, have incorporated it with Portland cement, sand and water, thus transforming it into a slab of artificial rock of a desired dimension.

In the construction of the Allentown-Easton road, every advanced engineering practice, every resource of the country through which the road passes, and the most modern mechanical machinery and appliances were utilized in the interest of perfect and economic construction. Every detail was given the most thorough study, so that only the best method of construction would be used and the results obtained give the highest type of work.

After the survey was made and the plans prepared, stakes were placed for grading and the actual construction of the road was started on July 9th. On account of lack of labor, rough grading was not started until July 18th, and the progress was limited until July 22d, when there were 50 men on this work. The same labor trouble prevented concreting being started until August 26th.

The entire length of the highway is paralleled by the tracks of the Lehigh Valley Traction Company. The major portion of the length of the road consists of an old dirt road, clay underlaid with limestone formation,

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and the topography mitigates against easy drainage. The old grades of the road were very irregular and "choppy," and due to the fact that there never had been any consideration given to the reduction or regularity of grades, the grading forms a considerable item in the construction cost, as the grade is established to true tangents and vertical curves. All fills were made in layers not exceeding 1 ft. in depth and rolled with a 10-ton 3-wheel road roller. Not only were the fills rolled, but the entire sub-grade was rolled until thoroughly compacted and true to grade and cross-section. Grade stakes were set every 25 ft. and the sub-grade was finished very accurately to the established grade and cross-section and all traffic kept off the finished sub-grade.

One of the most important principles involved in road building is that of thorough and proper drainage, and this principle was given the utmost consideration in the construction of this road. Corrugated iron pipe of sufficient sizes were placed wherever necessary to carry the surface water under the road and properly dispose of same, and concrete culverts were built to take care of the small streams. Under-drains of 4-in. porous tile, laid on ½ by 4-in. boards to maintain an even grade were placed along the entire length of the road, parallel with the road between the concrete and the trolley tracks and from 18 to 24 in. below the finished sub-grade line. All joints were wrapped loosely with a piece of one-ply tar paper with no lap at the bottom. This paper contains very little tar and will allow easy ingress for the water, and is considered more suitable than "bagging" which is generally used. The trenches were back-filled with crusher tailings or other stone crushed to a size of 3 in. or over. A similar tile drain was laid through all cuts, outside the concrete on the side opposite the trolley tracks. Latteral under-drains were placed wherever the conditions of the sub-grade warrant. All under-drains were carried to an outlet so as to properly dispose of the water and keep the sub-grade thoroughly dry.

All of the concrete work was economically arranged with reference to the handling and distribution of materials. Arrangements were made with the Traction Company for the transportation of all stone, cement, sand, forms, reinforcing, etc. The necessary tracks were laid into the local quarry at the Bethlehem end of the road, which was a commercial proposition, and also into the quarry at Farmersville, half way between Bethlehem and Easton, which was being operated as a day labor proposition. The railroad siding of the Taylor Wharton Company was extended sufficiently to place seven cars, and the trolley tracks were laid into and parallel with same. All sand and cement were received at this siding and transferred from the railroad cars to the trolley cars. All hauling was done in two shifts, the day shift handling stone, and the night shift, cement and sand. The stone from both quarries is a hard dolomite with a French coefficient of wear exceeding 14. The sand was obtained at Succasuna, N. J., and is dredged from banks underlying land adjacent to Lake Hopatcong, containing a negligible percentage of loam or dirt.

In addition to the customary small tools required for the construction of a road, the major equipment consisted of two rooter plows; two 10-ton

3-wheel power rollers; two No. 16 Koehring mixers with boom and bucket delivery; two 4 H. P. C. H. & E. gasoline pumps; four miles of 2 in. wrought iron pipe; two 100-ton crushing plants, and one small gasoline crusher for drain stone.

After the rough grading was done, the sub-grade was finished and rolled while the drains were being placed. The "concrete" crew, the "finishing" crew, and the "curing" crew followed in order, after which came the "trimming up" crew who removed the earth from the finished concrete surface and properly constructed the berms and opened the gutters. The men were distributed with care to insure uniform progress of each operation.

Special attention was given to the proper distribution of the concrete materials, and re-handling was practically eliminated. The sand and stone cars were dumped at measured distances from the end of the last placed concrete. The cement was stored in portable canvas-covered sheds holding ninety sacks each, placed at measured distances along the road, and carried ahead as they were emptied.

Thorough organization of the two concrete crews was effected. Each crew was organized with two finishers, two men handling the strike boards, two men placing concrete and reinforcing, one mixer engineer, one fireman, one charging man, two men on forms, three men on cement, eight men on sand and stone, and three men sprinkling and covering the finished surface. Each member of a crew having his particular task to perform each day, made him most efficient and secured the best results.

The road was constructed true to line, grade and cross section; 16 ft. of concrete with 3-ft. berms and 3-ft. gutters on each side. In many places, caused by the nearness of the trolley tracks, it was necessary to construct only a 2-ft. berm and 2-ft. gutter, while in other places, a 3-ft. or a 4-ft. berm was used, having a slope of 1 in. to 1 ft. with adequate surface and under-drainage. The sub-grade, which is flat transversely, was brought to a firm, non-yielding surface by rolling with a 3-wheel 10-ton power roller, and all portions which were inaccessible to the roller were thoroughly tamped. After the sub-grade had been prepared as described, to conform to the established grade, the materials for the concrete were distributed and the work of constructing the concrete roadway started.

The side forms, which are of 6-in. heavy Baker steel forms in 12-ft. lengths, were set true to line and grade and held in place by steel stakes so that the upper edges of the forms conformed to the established grade of the road. All mortar and dirt were removed from forms that had previously been used and the forms oiled before resetting them.

Immediately prior to placing the concrete, the sub-grade was thoroughly wet to assure proper curing of the bottom of the slab, but the water was not allowed to stand in pools. After being properly mixed, the concrete was deposited rapidly upon the prepared sub-grade in successive batches to a depth of $3\frac{1}{2}$ in. at the sides and $5\frac{1}{2}$ in. at the center for the entire length between the transverse joints; before the concrete could receive its initial set, reinforcings were placed the entire width and length between the transverse joints, and the remaining $2\frac{1}{2}$ in. in depth of concrete placed. This

entire operation was continuous, as no intermediate joints were allowed, and in case of a breakdown of the mixer, the concrete was mixed by hand to complete the section. Transverse joints were placed between each section.

The reinforcing used was No. 25 Kahn Road Mesh in sheets 5 ft. 2 in. wide and from 6 to 10 ft. long. It was laid parallel with the axis of the road, joints staggered, and with a lap of 4 in. crosswise and 12 in. lengthwise. The transverse joints were made by tying two 9-in. by 16-ft. strips of $\frac{1}{8}$-in. elastite to a 9-in. by 16-ft. steel plate, the assembled joints being placed across the pavement perpendicular to the sub-grade, at right angles to the center line of the road and 39 ft. 6 in. apart, with the elastite next the section being finished. Concrete was placed on both sides of it, that on the elastite side being stuck off, the strings cut and the plate pulled out by its handles. After the concrete had set 24 hours, the elastite was trimmed off $\frac{1}{2}$ in. above the finished section of concrete.

The concrete consists of one part Portland cement, two parts sand, and three parts crushed stone. The sand, being dredged, was very clean and graded in size from very fine to pea-gravel size.

The crushed stone, a dolomite, was of such size as to pass a screen having a $1\frac{1}{2}$-in. circular opening and retained on a screen having a $\frac{1}{4}$-in. circular opening.

By experimenting and testing, it was proven that the maximum strength of the concrete as proportioned above resulted from being mixed 90 seconds in a batch mixer of approved type, and continued for 17 complete revolutions of the drum for each 60 seconds. Accordingly, this method was followed and each batch was mixed 90 seconds, receiving from 24 to 26 complete revolutions of the drum. Sufficient water was used to produce a plastic concrete, so that when it was deposited, it would settle to a flattened mass but not cause a separation of the mortar from the coarse aggregate when handled. The drum of the mixer was completely emptied before mixing the next batch. No retempering of any batch which had taken its initial set was allowed.

The concrete roadway was constructed in sections 16 ft. wide, 39 ft. 6 in. long (having a transverse joint as above described, between each section), 6 in. thick at the sides and 8 in. thick at the center, thus giving a 2-in. crown from the center to each side of the road.

After placing the concrete as above described, the surface of same was struck off with an extra heavy strike board, followed by a lighter one, both of which were moved with a combined longitudinal and crosswise motion working off the side forms. When the strike boards were within two to three feet of the transverse joint, they were lifted to the joint and the pavement struck off by moving them away from the joint. Any excess concrete which accumulated by striking the pavement was removed and any holes left by removing the steel plate used in placing the transverse joints were immediately filled with grout.

After the concrete had been brought to the established grade and crown with the strike boards, it was finished with wood floats in such a manner as to properly compact it and give a surface free from all inequalities. When floating the concrete, the men worked from a bridge, resting on the side forms,

extending over the concrete, but not touching the concrete at any point. The concrete at the transverse joints was floated with a double float, allowing the elastite to pass through the center of it and thus producing a smooth, even finish on each side of the joint. The edges of the concrete at the sides of the road and at the transverse joints were beveled with steel edgers so as to eliminate chipping. A 2-in. radius edger giving a $\frac{3}{4}$-in. bevel was used at the sides and a 1-in. radius edger giving a $\frac{3}{16}$-in. bevel was used at the transverse joints. The floating and edging was not done until the surface of the concrete had a glossy appearance. After the floating had been completed and the concrete sufficiently set, the forms were removed, generally in about 18 hours.

Under a very hot sun, the finished concrete was protected by canvas covering supported by wood frames to prevent the canvas from touching the concrete. The finished surface of the pavement was sprinkled with water as soon as the concrete had set sufficiently to prevent pitting, and was kept wet until a 2-in. earth covering could be placed without damage to the concrete. This earth covering was kept wet for a period of 10 days and after 14 days it was removed and placed on the berms where needed, and all excess disposed of. During this curing period, when the temperature dropped in the daytime to 50° F., the pavement was sprinkled occasionally, but not covered with earth. All traffic was kept off the finished pavement for 28 days.

The work at all times was under the supervision of the Highway Department and all precautions taken to secure a perfect construction in every detail. In addition to the department supervision, an engineer and two inspectors appointed by the Association of American Portland Cement Manufacturers were present at all times and also had supervision as to the construction details. One of these inspectors was stationed with each mixer whose duty it was to see that all details were properly carried out as to the measuring of materials, timing the mixing by watch, placing the reinforcing and transverse joints and finishing the concrete.

On account of the scarcity of labor and the inability of the Traction Company to deliver the crushed stone as required, it was impossible to operate both mixers continuously. When the mixers were working properly, eight sections were laid and finished each day. The last concrete was placed on the road November 23d. The machinery and tools were then stored and the necessary repairs made preparatory for next season's work.

ESSENTIAL FEATURES FOR SUCCESSFUL CONSTRUCTION OF CONCRETE ROADS.

By William M. Acheson.*

This subject—the Essential Features for Successful Construction of Concrete Highways—is practically academic in which first principles are treated, and these at all times are the primary and necessary ones in road construction of any type.

First. Proper drainage of the roadbed is the primary principle, and the oversight or neglect of this feature frequently means failures and increased cost of maintenance which can only be remedied by re-construction and the proper drainage facilities installed.

The system of drainage which is to be installed or developed depends entirely upon local conditions. Deep side ditches parallel to the axis of the road are very essential to the successful life of a highway for the reason that these side ditches always insure quick drainage of the road prism.

Where porous tile is used it should be laid under the road in order to take care of the excess drainage conditions which cannot be handled by side ditches. Where this tile is laid parallel to the road it should be placed from two to four feet below the finished grade line, and have frequent openings in order to freely discharge the water it carries, and the tile should never be less than 4 in. in diameter.

For carrying this water ditches should at all times be constructed to insure a quick discharge from the section of the road.

Road drainage comes under two heads: surface drainage and sub-surface drainage.

With the drainage problem properly designed, next comes the preparation of the sub-grade; this sub-grade should be studied for all deficiencies in connection with the character of the soil, poor drainage and the alignment of the road with reference to side hill construction. The sub-grade should be rolled until it is firm; all soft and unstable places excavated and replaced by satisfactory material.

The sub-grade as finally prepared should be compacted and firm, and in such a state that the construction work which is necessary to carry on over the sub-grade will in no way injure or destroy this finished sub-grade; by that I mean that the contractor's equipment and materials which are used could be upon the finished sub-grade and his work carried on, and that his crew would not find it necessary to replace any part of this sub-grade.

Foundation Course or Sub-Base.

Where the soil conditions develop and they cannot be corrected by replacing more stable soil, sub-base should be laid. From a study of the

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highways which were built in Division 9 during the past two years, I have found that the following type of foundation course the best for cement concrete pavements:

After the necessary excavation has been made and properly drained, a layer of clean gravel from 2 to 4 in. in thickness is spread over the bottom. On this place a quarry or field stone course of the required thickness, depending upon the soil conditions.

After the voids have been filled with clean sand or fine gravel and consolidated, it should be covered with at least 2 in. of coarse sand. This last mentioned sand layer serves the double purpose of regulating the sub-grade and acting as a cushion to take the impact off the pavement proper.

Concrete roads should be reinforced with a light mesh reinforcement when bad subsoil conditions are encountered. We are using reinforcings every place there is any doubt, and sometimes use it in conjunction with the foundation course, and also thickening of the slab.

One of the essential features of the reinforcing is to be sure the reinforcement is in its proper location as to cross-section.

It has been our practice to lay the first course of concrete and on this place the reinforcement, usually about 2 in. below the surface grade by the use of wire hooks; it is then covered with the necessary 2 in. of concrete and struck off to the finished section.

I might bring in here a few features in connection with workmanship.

In the preparation of the sub-grade great care should be taken by the contractor to see that at all times proper lateral ditches run from the sub-grade to insure quick drainage after a rain-storm, for where water lays on the sub-grade it sometimes develops a soft spot which is often overlooked by both the engineer and contractor, on account of the thoroughness with which the work was done originally and left as finished.

Another point I wish to emphasize is the necessity of any system of underdrainage or foundation courses after the culverts are completed and the proper surface drainage provided.

Underdrains and foundation courses are used to increase the bearing power of the soil, and the sooner they begin to perform this function, the better the results for the highway.

In the above features, which are fundamental, I have tried to cover drainage, grading or preparing a firm base, and the necessity of having the sub-grade conform in every detail to the alignment and section which are known to be the best practice.

In the sub-grade it has been demonstrated that a flat sub-grade is the ideal type for a concrete road, and this is the reason that lateral trenches from the sub-grade to the ditches are not only necessary during the construction period, but can be used later as side drains to aid in keeping the foundation course dry.

MATERIALS.

Hard and fast rules should be laid down in regard to the material in the construction of the cement concrete pavements.

These are also the main factors in the successful construction of any highway, and should never be deviated from.

The sand should be clean and coarse graded. The coarse aggregate should be clean, hard, tough and properly sized, and the cement that which meets with all the requirements of a high grade Portland cement.

In this detail I may differ from some engineers. I mean by this that if in certain localities the proper quality of material is not available without importing, and this is prohibitive on account of the resulting high cost, I may say right here, do not attempt to build a concrete road.

The question of proper coarse and fine aggregate is paramount in cement concrete road construction.

Mixing.

The proper mixing of concrete has been developed by the invention of modern machines, and the Portable Batch Mixing machine is the most economical and the proper one to be used on concrete roads, and the "batch" type is the one that is universally used as against the "continuous" mixture.

I believe the most important feature of mixing concrete is, first, the number of revolutions based on the time which the mixer turns in order to turn out a batch of concrete. Second, the water. I place the number of revolutions first for the reason that by giving the mixer a few revolutions and excess of water, you would have the appearance of good concrete, but by giving the required amount of revolutions which practice and experiments have shown to be necessary, the water situation solves itself, and I believe in this connection that the minimum revolutions of the drum should be twelve and the minimum length of time fifty-five seconds.

The consistency of the concrete as turned out from the mixer may be best described: The concrete should be a "quaky" mixture; not wet enough for the mortar to run away on the sub-grade, but plastic enough to be easily worked. It is absolutely necessary to procure this result that the correct quantity of water be determined and its use of volume practically constant, and the condition of the aggregate will mean, in some cases, the change of the volume of water. Care is always necessary that the aggregates are accurately measured. These are the essential features of good concrete construction.

In the selection of the type of pavement, there are three very essential points. First: The traffic which it is required to carry. Second: The money which is available for the construction of the pavement, based on the mileage which should be built to get satisfactory service in handling the trade traffic required for the district. Third: The annual cost for repairs should at all times be taken into consideration in the final selection of the type of pavement, and that *third* point when given due consideration has demonstrated that what sometimes looks prohibitive from a importing standpoint is not so; and it also brings out another point which is more true of the design of highway work than any other I know of—not enough detail research is made during the preliminary technical study of a highway.

In New York State in the southwestern part, which is known as the lower

tier, are sections which have been known for the scarcity of sands of the proper quality.

In this section there were designed a number of gravel concrete roads with a thin bituminous top, which were complete failures. A material survey was made of this territory by the engineers, with the assistance of the Bureau of Research, and sand was found which passed the requirements by being washed.

To have the commercial sand and gravel companies install washing plants was a task, but this was finally done, and it was not long until a contractor on a highway installed one for himself.

I mention this as an instance where research sometimes surmounts an obstacle by opening up a certain section of territory to the building of concrete roads which otherwise could not have been constructed on account of the high cost of importing materials.

The practice in the State of New York, and which has been demonstrated to be successful, is $1 : 1\frac{1}{2} : 3$ concrete.

The fine aggregate shall be sand, free from organic matter; that which shows a coating on the grains shall not be used until satisfactorily washed, and shall have the following graduation: 100 per cent shall pass a $\frac{1}{4}$-in. screen; not more than 20 per cent shall pass a No. 50 sieve, and not more than 6 per cent shall pass a No. 100 sieve. In special cases where more than 20 per cent of sand passes a No. 50 sieve, and the sand is well graded to give a low percentage of voids, it is allowed to be used; sand is rejected if it contains more than 5 per cent of loam or silt.

Mortar in the proportion of one part of cement to three parts of sand shall develop a compressive strength at least equal to the strength of a similar mortar of the same age composed of the same cement and standard Ottawa sand.

If gravel is used it shall consist of clean, sound, tough, hard stone.

Broken stone shall have clean and sharp angles, and shall pass the standard tests, as adopted by the American Society for Testing Materials.

The sizes of gravel we are using at present is that which will pass through a $1\frac{1}{4}$-in. circular hole and that will not pass through a $\frac{1}{4}$-in. hole. The broken stone must pass a $2\frac{1}{4}$-in. circular hole, and that will not pass through a $\frac{5}{8}$-in. hole.

On highways where local stone is being crushed, I think up to 10 per cent of the stone passing a $\frac{5}{8}$-in. screen and not passing the $\frac{1}{4}$-in. screen, should be used in order to get the full benefit of the local material.

In the installing of joints, whether a pre-molded joint, a wood joint or a steel joint, great care should be taken that the joints reach to the sub-grade in the case of the three types, and in the wood and pre-molded joint it should project at least $\frac{1}{4}$ in. above the finished pavement, and a split float used to insure the plane being continuous to the slabs. These joints may then be cut off to conform to the section of the road.

In the case of the steel joint, great care should be taken to see that the camber of the steel joint and the camber of the screed used are identical, and these joints should at all times be laid perpendicular to the axis of the road and perpendicular to the plane of the road.

Unless these joints are carefully installed as above there will be a tendency for one joint to slip against the other, especially on grades.

FINISHING AND CURING.

The surface of a concrete road should be screed by means of a template at practically right angles to the axis of the road, with a short, sawing action. A heavy screed should be used, and at the same time it should not be necessary to move this screed over the concrete surface more than once; following this should be the smoother. This is frequently referred to as a float, but the word float is a misnomer. The real intention is to smooth the surface so it will conform to the finished section. This smoothing tool should at all times be made of wood.

Sprinkling with hand pots should be started as soon as the concrete surface will stand it. In this case I mention hand pots for the reason that the pressure from a hose on the first sprinkling is usually too severe on the surface; this should be done from one to two hours after the concrete is placed. In this connection it is very essential that this hand sprinkling should be done several times before sod or earth protection is placed over the surface, and then after it is placed should be sprinkled at least twice during the day and once at night. In the curing of concrete roads, judgment and common sense enter into this feature very strongly, for with hot suns and drying winds to contend with, unless the curing is properly looked after a serious damage will be done to the road surface.

Under favorable conditions a concrete road should be closed to traffic from ten to fourteen days, and in the fall or a slow-curing period it should be kept closed three weeks.

CONSTRUCTION.

The type of pavement that is standard in New York State is known as one-course road, and this is really adopted for the reason that we believe a one-course road is much more successful while it might be a little more expensive, and it is also a more simple type of road to construct than a two-course road, for the reason that it is a one operation type of pavement. The two-course pavement probably could be used where you might have local material which would be satisfactory for the construction of a foundation course and the top or wearing course would have to be of imported materials.

I doubt very much that this factor of cost should be given too much weight or preference in the building of a concrete highway.

Of course, in a concrete road it is necessary that forms should be used, and the successful and economical type of form is either a steel form or a wooden form with angle irons on. The angle iron on a wooden form is necessary and economical, as the top of the wooden form becomes rough and tends to give an uneven pavement, and the angle iron on a wooden form absolutely insures against an irregular surface; these forms should be set true to line and grade.

Joints in the New York State specifications are provided every 30 ft. The reason for the adoption of 30-ft. joints is a practical one.

There have been concrete roads built, and an inspection made of the same after several years it was found that the transverse cracks which occurred in the pavement averaged 30 ft. apart, and later inspections have demonstrated that this was a good decision, although we are in the experimental mood and probably will try some at greater distances.

On the roads which have been constructed in New York State, with one exception, the material has been delivered on the finished sub-grade in windrows, the stone on one side and the sand on the other; care has always been taken that these materials should never become mixed before being proportioned for the mixture.

An improved type of mixer is used and the batches have been from 2 to 4 bags to the batch, the bag being generally taken as the basis of the loose material.

The organization has generally been, one man sprinkling and throwing on cover, one man smoothing, two men on the screed, three men in front of the mixer spading the concrete to conform as nearly as possible to the section required, and two operators on the machine—one on the supply bucket and one depositing the concrete.

Behind the machine from 15 to 20 men, depending absolutely upon the life which the foreman in charge instills into his gang.

The materials are so distributed that the wheeling distance is as short as possible at all times, and by this I mean 50 ft. from the mixer.

WORKMANSHIP AND RESULTS.

Under this heading is covered tools, which should all be modern and thus insure the best results.

I believe in a heavy split float for taking care of the joints, and a long-handled float which enables the operator to always work in a normal position over the bridge, and at the same time eliminates an additional man, which is necessary when hand floats are used.

Under the heading of results, I wish to mention that good concrete depends to a very great extent to the proper mixing of the materials. You cannot get good concrete unless the concrete is properly mixed. It has been demonstrated to our satisfaction that it is necessary to give the mixer drum at least twelve revolutions.

Materials as called for in our specifications and mixed twelve revolutions have shown an average compression test for $1 : 1\frac{1}{2} : 3$ mixture 28 days old of 3500 lb. per sq. in., and in many cases have shown compressive strength of 5400 lb. per sq. in. for the same period of time.

The material as coming from the concrete mixer should be deposited so it is of such consistency that the mortar does not run away from the coarse aggregate.

The material as deposited is handled by three men in front of the mixer, in order to make the screed do as little as possible.

The finishing should always be within 10 to 15 ft. of the screed, and this is impossible if the concrete it too wet.

At the joints a split float should be used in order to insure the road having a continuous plane for the finished pavement.

I advocate the brooming of a concrete road for the reason that the marks which the broom makes in the pavement aids in holding the moisture longer during the curing season.

Care should be taken, however, to see that the broom marks are not too deep so as to start spalling and impede the surface drainage.

Our roads are covered with loam, sod or any soft material and wet down twice a day in the hot summer. In the fall we have constructed pavements on which we placed no cover at all, but were wet down at least once a day.

One of the most important parts of a contractor's road plant in the construction of cement concrete roads is the water supply. I would advise at least a 5-horse power engine with a pump capacity of at least 200 gallons per minute.

The pipe line should be at least 2-in. pipe, and should have tap at least every 200 ft.

Wire-wound hose should be provided long enough to reach half way between the taps. A great many mistakes have been made in the past in regard to this particular feature.

I have endeavored to cover in this paper the principles of good construction, design, materials, organization and equipment in constructing a concrete road.

In closing this I want to say that every highway constructed of different types is absolutely a reflection on the individuality of the contractor and the engineer on the road, and for which they are responsible for either the success or failure of the highway as to design, materials and workmanship.

CONCRETE FOUNDATIONS FOR ASPHALT PAVEMENTS AND ROADS SUBJECT TO HEAVY TRAVEL.

By Clifford Richardson.*

The first essential in road construction of a permanent character is a dry and firm subsoil upon which to build. This being assured, another and more essential factor in the satisfactory construction of the highest type of asphalt pavement, country highways, and, in fact, for all forms of road surfaces to carry heavy traffic, is a suitable foundation. This is emphasized by the weights which our main arteries of communication are called upon to sustain owing to the advent of the motor vehicle, more especially the motor truck, the latter rapidly increasing in capacity to such an extent that means for regulating its use are being considered. Portland cement will, therefore, find an extended use as travel on our roads increases, for the purpose of affording adequate support for any type of surface. The only satisfactory substitute therefor has been found to be an old, well-compacted broken stone road which, at least in the United States, is not often available, but of which good examples are found in England.

In the case of asphalt surfaces these, in themselves, possess no inherent strength to support heavy loads and, like all other surfaces, require an adequate support when subjected to concentrated travel. Whatever type of pavement or road surface, of a character to carry heavy travel, may be selected it must, unquestionably, be given such support by a proper foundation, and this can be accomplished in no more satisfactory way than by a suitable Portland cement concrete. While this has been recognized for some years, in certain instances, it is plain that it must have a general acceptance in the near future, at least as applied to our main arteries of communication, while it is generally desirable as a matter of permanence, where funds are available, in view of the much longer life which any form of construction will have if placed thereon. The primary difficulty is, of course, the availability of funds for this purpose. As a well constructed Portland cement foundation needs no maintenance its cost may be met by the issue of long term bonds where such a procedure would violate all principles of economics as applied to the surface itself, the life of which, of any type, cannot be such as to justify a bond issue for a longer period than ten years.

The thickness of concrete which is called for as an adequate support for a road or street is, of course, dependent upon the character of the travel it will be called upon to carry and upon the support which it receives from the sub-soil on which it is placed. In London, England, on one of the streets of the heaviest traffic, High Holborn, it has been proposed to place 11 in. as being a thickness necessary for the conditions to be met there. The usual thickness in this country for our most important thoroughfare has been

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6 in., but with the enormous disturbance which is continuously going on for underground operations and with the increasing weight of the motor truck this will, in the near future, have to be increased in proportion.

At the same time it is becoming recognized, at least by experts, that there must be a great improvement in the methods employed in the selection of the aggregate, especially the sand, of which concrete is made. Testing the sand is as important as testing the cement if work of the highest character is expected. It has also been demonstrated that one brand of cement will give more satisfactory results with one sand than with another, although each cement will yield the same results with standard sand. It is evident, therefore, that there is a great opportunity for improvement in the character of concrete and it is encouraging to know that the United States Office of Public Roads and Rural Engineering and several specialists on the subject are making extended investigations in this direction.

A Portland cement concrete foundation may be looked upon as a permanent investment and, to a certain extent, concrete road surfaces may be also considered in the same way, as when they become too rough with age to be used as such they will, with adequate repairs, become available as a foundation for an asphalt surface or one of some other type.

California, under the policy pursued by its State Highway Commission, is expending millions of dollars in the use of Portland cement concrete for highways, and looks upon this expenditure as an asset for many years. Upon this concrete an asphaltic surface, composed of sand and fine pea-grit, is laid for the purpose of obtaining one which is more agreeable for travel at a cost of but 8 cents per sq. yd. This is a temporary expedient where the travel is light and funds are not available for a more suitable form of construction. It is expected to last for three or four years and give an adequate return for the money expended. Where travel is more concentrated, an asphalt surface mixture containing fine stone is employed, costing about 50 cents per sq. yd., which may be looked upon as having a higher degree of durability. Probably there is no state where as satisfactory a combination of Portland cement concrete and bitumen has been used, looked at from the point of view of economics, as in California, although it may be regarded as too thin, having a thickness of only 4½ in. This is, of course, due in part to the favorable climatic conditions which the form of construction employed there is called upon to meet.

It is worthy of note that where Portland cement concrete is protected by a bituminous or other form of surface, a much smaller proportion of the hydraulic binding material and a lean concrete is called for, than where it is exposed to travel and is, consequently, much less expensive.

Experience has shown that our present methods of mixing and placing concrete for road construction may be improved to a very large extent. Too much work of this description is carried out carelessly, without a proper study of the mineral aggregate of which the concrete is composed and without devoting sufficient time to its thorough mixture, but engineers are awakening to their shortcomings in this respect and there will no doubt be a great improvement in this direction in the future.

Of course, the characteristics of the Portland cement in use in concrete foundations for road surfaces and the degree of fineness to which it is ground, form an element of great importance in the success with which concrete is employed for any purpose, although it is recognized that the product of our American industry is generally of a very high quality.

Climatic conditions and the great extremes of temperature, at times ranging from 130° to 40° F. below zero, result in the formation of cracks in Portland cement concrete when used as monolithic foundations for asphalt surfaces, and these are often reproduced in the surfaces themselves, but the effect of such cracks usually disappears within the first few years of the life of the surface and are not subsequently manifested to any disagreeable extent. That these cracks are largely due to contraction, due to extreme temperatures, is demonstrated by the fact that they are far less apparent in such a climate as that of California where extreme temperatures do not occur. In certain instances expansion of Portland cement concrete, at joints between different day's work, have been observed, but this happens but rarely and, again, is not manifested after the expiration of a few years' time.

FOUNDATIONS FOR PERMANENT PAVEMENTS.

By R. C. Stubbs.*

A marked structural peculiarity of even the thickest street pavement is its thinness compared with its area and the fact that surface earth is used as its foundation. I have observed that this peculiarity results in damage to a residential street pavement from forces acting in the earth and from weather, as well as from traffic. At the 1914 convention of the Institute I presented a paper on "The Over-Compression of Subgrade" in which attention was called to the effect of unequal rolling of the subgrade and over-rolling at the center of the street. Attention was also called to the high compressiblity of certain clays and their eventual return to their original volume, lifting the pavement slab by this movement. Where the foundation area has not been uniformly consolidated by rolling, the upward pressure of the earth as it resumes its original condition varies from place to place, and this variation may combine with the traffic loads to produce very trying stresses, which may be called "earth stresses." The purpose of the present paper is to point out another class of earth stresses which was not discussed in my previous paper.

The earth foundation of a pavement would remain in a uniform condition as respects moisture if it were not for altering wet and dry seasons. In any case the earth under the center of a pavement slab will remain in a practically uniform condition, assuming that the roadbed is properly drained. During a dry season the exposed earth in the parking back of the curb on a residential street becomes porous and sometimes crossed with small cracks. As the dry weather continues, this condition extends down through the earth, under the curb and gutter, and finally under the edges of the pavement slab, under which it may continue during a prolonged drought as far as the quarter points, as indicated in Fig. 1. As the earth dries out it shrinks and leaves the edges of the pavement slab without as much support as is afforded to the central strip of the slab. When the rainy season begins the very dry earth furnishes ample opportunity for water to enter the pores and minute fissures of the foundation of the outer parts of the pavement slab. In some soils this sudden absorption of water will cause the foundation to swell, tending to lift the slab at the sides, as shown in Fig. 1.

If the pressure or reaction of the earth against the bottom of the pavement slab is called zero for normal conditions, neither too wet nor too dry, there will be a minus pressure or reaction under the sides of the slab in dry weather and a plus pressure in wet weather. Along the quarter points of the street there are neutral strips where there is a tendency for the zero pressure condition to remain permanently. Here the slab will have tension in its upper parts during the dry season and during the wet season the tensile

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stresses will be found in the lower parts of the section, as is evident from a consideration of Fig. 1. This explains the formation of some of the longitudinal cracks at the quarter points of street pavements.

If the over-compressed portion of the foundation is a comparatively narrow strip along the center of the street, the stresses due to the return of the earth to its normal condition after over-rolling, combined with those due to dry-weather conditions may cause enough tension near the center of the slab to produce longitudinal central cracks, particularly if the effect of heavy travel loads on the central strip of the pavement during previous wet weather is considered.

It is obvious from this discussion that it is desirable to keep the moisture content of the earth foundation as nearly uniform as possible. As it is reasonable to assume that the moisture under the center of the pavement slab remains fairly uniform, the problem is to spread this condition throughout the entire foundation, including that of the gutters and curbs. This can be done with either a plain curb or a combined curb and gutter as shown in the lower part of Fig. 1. The remedy is to carry downward a thin extension wall, what dam builders call a "cut-off," so that the dry weather cannot affect the foundation of the pavement slab and the latter will have a uniform support over its entire area, except for the effect of over-rolling at some places. If the depth of the standard curb is 19 in., that of the curb and cut-off should be not less than 30 in.

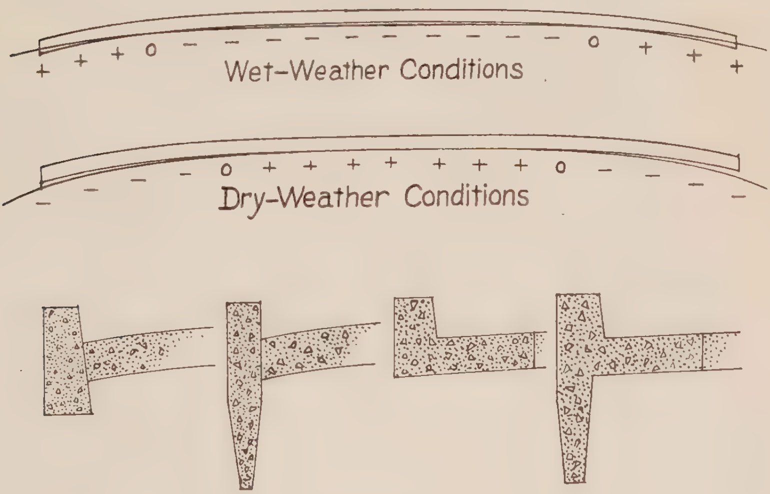

FIG. 1.

The standard curb and gutter is not deep enough to prevent displacement by weather conditions, and during dry weather a seam opens between it and the pavement slab which allows the rain water of the wet season to

enter the earth of the foundation by a short path. The concentration of a large part of the weight of the combined curb and gutter along a comparatively narrow strip of the foundation also tends to cause such a seam or crack. It is evident from the illustration that a cut-off adds materially to the stability and freedom from canting of such combined curbs and gutters.

These deep curbs I have found to cost but little more than the standard types. If a standard curb is 19 in. deep, 6 in. wide at the top, and 8 in. wide at the base, the cut-off curb is made 5 in. wide for a depth of 18 in. and then tapered to a width of 3 in. at a depth of 30 in. The excavation for the upper part is carried on in the usual way and the forms placed; then the narrow excavation for the cut-off is made with a special spade.

PROGRESS REPORT OF COMMITTEE ON SIDEWALKS AND FLOORS.

The Committee on Sidewalks and Floors, appointed September 20, 1915, has prepared as an outline of the work it contemplates doing a comprehensive program as follows:

Foundations.—Character of soil and local conditions requiring sub-base. Character of soil and local conditions requiring no sub-base. Preparation of foundation. Drainage of foundation.

Preparation of Sub-base.—Material to be used in construction of sub-base (cinder, slag, broken brick, etc.) and methods of compacting same. Depth of sub-base as dependent on variations in temperature and foundation conditions.

Concrete, Two-Course Work.—Thickness of upper and lower course. Proportions of concrete for lower course. Proportions of concrete for upper course.

Concrete, One-Course Work.—Thickness of concrete slab. Proportions of concrete for one-course work.

Fine Aggregate.—Character of material, grading and size.

Coarse Aggregate.—Character of material; grading and size for upper and lower course of two-course work, and for one-course work.

Mixing and Placing.—Mixing and placing concrete for one and two-course work. Proper consistency to avoid segregation of aggregates. Precautions during cold weather. Coloring matter, if used.

Finishing.—Methods for securing best results in one and two-course work, with suggestions as to proper tools to be used.

Reinforcing.—Special conditions under which reinforcement should be used, including matters of foundation and temperature. Size of metal required for reinforcing. Position of metal with reference to top or bottom of slab.

Joints.—Spacing of expansion joints. Width of opening of expansion joints. Thickness and character of material to be used in expansion joints. Cutting concrete sidewalks into slabs or sections.

Protection.—A, Traffic. B, Weather.

Failures.—Due to poor foundations. Due to improper mixtures. Due to improper materials. Due to heaving from temperature, moisture, or other causes. Due to improper finishing. Due to dusting. Prevention of failures. Remedies to be applied to existing work which has totally or partially failed in order to put it in serviceable condition.

Through extensive field investigations and correspondence the committee hopes to secure valuable information on all of the points noted and trusts that the co-operation of the members of the Institute will be given in supplying such data as are requested of them.

The committee also invites criticisms and suggestions on the above outline of its proposed activities. After a through consideration of the material the committee is able to collect, certain revisions and changes will probably be suggested in the present specifications for sidewalks and floors.

It is also hoped that a recommended practice for construction will be compiled.

Your committee suggests the advisability of considering the combining of the Committee on Concrete Roads and the Committee on Sidewalks and Floors, dividing the work of such combined committee into sub-committees in ways which will effectively produce the desired results, and which will at the same time harmonize specifications and recommended practices on these subjects where such specifications and practices contain clauses of a general nature.

LEWIS R. FERGUSON, *Chairman.*

REPORT OF COMMITTEE ON TREATMENT OF CONCRETE SURFACES.

This committee offers for the consideration of the Institute the following recommendations:

First.—Inasmuch as lump lime and hydrated lime are permitted in the standard specifications for Portland cement stucco on metal lath, brick, tile or concrete block, and for Portland cement stucco on wood lath, that the Institute adopt as their standards the specifications for lump lime and hydrated lime of the American Society for Testing Materials.

Second.—That the Institute's standard specifications for Portland cement stucco on metal lath, brick, tile or concrete block and the standard specifications for Portland cement stucco on wood lath be revised as follows:

Standard Specifications for Portland Cement Stucco on Metal Lath, Brick, Tile or Concrete Blocks.*

3. *Lime.*—The lime shall meet the requirements of the standard specifications for lump lime and for hydrated lime of the American Society for Testing Materials. The lime shall be thoroughly hydrated either by the manufacturer or the contractor. If hydrated by the contractor, it shall be slaked in sufficient water to make a soft paste and allowed to stand at least one week before being applied to the wall.

8. *Measuring Proportions.*—Methods of measurement of the proportions of the various ingredients, including the water, shall be used which will secure separate uniform measurements at all times. All proportions stated are by volume. A bag of cement (94 lb. net) shall be assumed to contain 1 cu. ft. When lump lime is measured by the contractor it shall be measured in the form of putty. Hydrated lime may be measured dry.†

17 (*a*). Galvanized or painted $\frac{1}{2}$-in. crimped furring, not lighter than 22 gage, or other shape giving equal results, shall be fastened direct to each studding, using $1\frac{1}{4}$ x 14 gage staples, placed 12 in. apart.

17 (*b*). Galvanized or painted $\frac{1}{2}$-in. crimped furring, not lighter than 22 gage, or other shape giving equal results, shall be fastened over the sheathing paper and directly along the line of each of the studs, using $1\frac{1}{4}$ x 14 gage staples, placed 12 in. apart. The same depth of furring should be adhered to around curved surfaces, and furring shall be placed not less than $1\frac{1}{2}$ in. or more than 4 in. on each side of and above and below all openings.

20. *Application of Lath.*—Place lath horizontally over the furring, driving galvanized staples $1\frac{1}{4}$ x 14 gage 8 in. apart over the furring into the studding.

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* The present specifications are printed in the *Journal* of the American Concrete Institute of October-November, 1914, page 38. The clauses given in the report are to be substituted for clauses having the same numbers in the present specifications.

† If hydrated lime is not thoroughly hydrated it shall be reduced to putty before measuring.

The sheets of lath shall be locked or lapped at least 1 in. and tied at joints between studs both vertically and horizontally with 18 gage galvanized wire.

23. *Brick, Tile or Cement Block Surfaces.*—*Existing surfaces* to be stuccoed shall have all dirt, dust or other foreign matter removed by means of a wire brush, whiskbroom or other effective means. Brick surfaces shall have all loose, friable or soft mortar removed from the joints to a depth of not less than ½ in. In case the surface has been painted, is oily or otherwise in condition that the stucco will not firmly adhere, then metal furring and lathing shall first be applied.

New surfaces shall have ample roughness to assure a strong bond and key between the stucco and the surface. In brick walls the mortar joints shall not be less than ⅜ in. thick and the mortar shall be omitted from or raked out of the joints for at least ½ in. back from the face to which the stucco is applied. Before placing the scratch coat the surface shall be clean from all dust, dirt or other loose particles and thoroughly wetted, and shall be in this condition when the mortar is applied.

24. *Plaster.*—(*a*) The *first coat* shall contain not more than two and one-half (2½) parts of sand to one (1) part of Portland cement by volume. If lime is added it shall not be, when hydrated, in excess of one-third of the volume of the cement. Hair or fiber may be added in sufficient quantity to bond the mortar.

(*b*) The *first coat* shall contain not more than two and one-half (2½) parts of sand to one (1) part of Portland cement by volume. If lime is added it shall not be, when hydrated, in excess of one-third of the volume of the cement. No hair, fiber or similar material of any kind or in any quantity shall be added to the mortar.

For *second coat,* the proportion of sand to cement shall not be greater than 2½ to 1 by volume, nor shall more than ⅓ part of lime be added.

For *third coat,* the proportion of sand to cement shall not be less than 2 to 1 nor more than 2½ to 1, by volume, nor shall more than ⅓ part of lime be added.

Standard Specifications for Portland Cement Stucco on Wood Lath.*

3. *Lime.*—(Same as 3 above.)

8. *Measuring Proportions.*—(Same as 8 above.)

23. *Plaster.*—(Same as 24 above except that paragraph *b* is omitted.)

Your Institute has during the past year been represented by the chairman and two members of this committee on an advisory committee appointed by the Bureau of Standards of the U. S. Government to assist in conducting an extensive series of tests of stucco of various kinds applied to various surfaces.

This series of tests is in reality an outgrowth of tests on the corrosion of metal lath embedded in stucco (*Journal* Am. Conc. Inst., Vol. II, page 445

* The present specifications are printed in the *Journal* of the American Concrete Institute for January 1915, page 83. The clauses in the report are to be substituted for clauses having the same numbers in the present specifications.

Nov., 1914) which was conducted on panels erected in 1911. Inasmuch as these first panels were of small size, it was decided to conduct these later tests, extending the scope to include all bases used in practice and a number of the most generally used types of stucco mixtures.

The structure bearing the test panels is a specially constructed building approximately 200 ft. long, 26 ft. wide and 25 ft. high, of two stories, covered with a low hip roof. Level ground not being available, the structure was located on a hillside, the south end overhanging the slope of the hill and being supported on a steel frame foundation resting on concrete footings.

The plan of the test structure calls for a total of 56 panels of which 22 are masonry panels located on the lower floor. The upper panels, together with the end lower panels and two lower side panels containing door openings, are all of bases other than masonry, viz: wood lath, expanded metal lath, wire lath, plaster-board and patented stucco-board, making 34 of these panels. The masonry panels include all types commonly employed in practice, brick, clay tile, concrete block, monolithic concrete and gypsum block.

Each panel is separated from the one adjoining by a wood pilaster and is provided with a small window, with the exception of the two panels on the east side in which are placed the two doors giving entrance to the interior of the building. The size of each panel is approximately 10 ft. high by 15 ft. long.

The total thickness of the stucco coats on the lath panels is $1\frac{3}{8}$ in. from the surface of the sheathing to the surface of the final coat. Metal lath is applied over $\frac{1}{2}$-in. crimped metal furring strips, and wood lath over furring consisting of lath laid vertically over the sheathing paper. In the double or counter-lathed panels, the wood lath is laid diagonally across the sheathing in two directions.

In order that the thoroughness of embedment of the metal lath could be determined, cut-out portions of the sheathing were provided to be afterward removed for making inspections. This of course does not apply to the back-plastered metal lath panels.

Since it was desired that the tests approximate as nearly as possible the conditions that obtain in ordinary building construction, the plastering was done by experienced local workmen under the supervision of a Washington contractor. No effort was made to enforce any undue precautions or to execute the work in a way which would be impracticable for ordinary commercial work.

The stucco surfaces are all finished in the same manner, a moderately smooth "sand float" finish, obtained by the use of an ordinary wooden float.

Full details of these tests, together with a progress report will be issued by the Bureau of Standards in the course of two or three months, in "Technological Paper No. 70."

Your committee proposes during the coming year to prepare for your consideration a Standard Recommended Practice for Stucco Construction. Suggestions from members or instructions from the Institute as a body, which will assist the committee and guide it in its work, will be welcomed.

Cloyd M. Chapman, *Chairman.*
H. von K. Borchard, *Secretary.*

DISCUSSION.

Mr. Tubesing. MR. TUBESING.—Last summer, in remodeling St. Paul's Mission in Milwaukee, we found the stucco and lath in good condition but only about half of the nails were of use in holding the lath in place. The remainder had corroded where the lath and studding came together. This indicates that the quality of the galvanizing should be specified, for it is only a question of time when such nails will become corroded.

Mr. Chapman. MR. CHAPMAN.—If the nails mentioned by Mr. Tubesing were examined it probably would be found that they were sherardized and not galvanized. Sherardizing is notoriously ineffective as precaution against weather exposure.

Mr. Collings. MR. COLLINGS.—In order to have durable work, even with fairly good galvanized lath, it is necessary to have the metal thoroughly covered by the plaster, and galvanized nails must be well protected, also, for there is a limit beyond which it is unreasonable to expect a protective covering to remain effective.

Mr. Tuller. MR. TULLER.—In making some tests of stucco a few years ago, we employed first-class plasterers to make the panels. In their anxiety to give us a very good job they did too much work on the panels and the result was a large number of crazing cracks. The work did not equal commercial stucco.

Mr. Johnson. MR. JOHNSON.—We recently examined a large number of stucco houses and found them all cracked, whatever the base that carried the stucco. It seems practicable to build stucco houses that will not crack but to do so the steel must have within its elastic limit the same strength as the slab of plaster. We have tried to work out this idea during the last two years but are not yet ready to report. There is necessarily some movement between the wood and the stucco in such houses. When wood is heated it shrinks and the percentage of shrinkage is much more than the coefficient of expansion of steel and in the opposite direction. Hence there must be a little movement between the stucco and the frame. Using the theoretically correct amount of steel will not prevent this movement, although it may prevent cracks in the stucco, and some provision must be made for it at the corners of the house, as by attaching the lath to the stud next the corner rather than to the corner stud.

Very few people notice the cracks in stucco buildings in which they live. A contractor said there were no cracks in such a house he had just finished and when a 10-ft. vertical crack near the front entrance was pointed out, he attempted to prove it was merely a surface crack by cutting away the material on the right-hand side of it with a hammer and chisel. The chips flew off, always from the right-hand side of the crack, until the plaster was cut through. This crack, which could not be seen more than 10 ft. away, is typical of a very common class.

Mr. Wig.—In the stucco test structure of the Bureau of Standards, which is 25 by 200 ft. in size ,there are many panels having very many irregular cracks which are not evident 10 ft. distant unless the stucco is wet. Although cracked the stucco so far may be said to be giving satisfaction, but it is apparent that the metal lath in such a structure must be more or less exposed. **Mr. Wig.**

President Wason.—Unless there is some objection, the report of this committee will be printed and the committee continued. (No objection was raised.) **President Wason.**

REPORT OF COMMITTEE ON SPECIFICATIONS AND METHODS OF TESTS FOR CONCRETE MATERIALS FOR 1916.

In the report of last year was described the co-ordination of the work of your committee with that of C-9 of the American Society for Testing Materials. As a result of the joint action thus made possible, the work has been extended and broadened, and funds have been provided for research.

The most important development during the year has been the establishment of the research laboratory at the Lewis Institute, Chicago. A chemist of extended experience and thorough training, Dr. Oscar E. Harder, has been engaged, and is prosecuting the studies and tests required for the investigation of the impurities in sands which produce defective concrete and of the methods of correcting them. This research, together with the other investigations under way, have been made possible by generous contributions of money and materials.

Various laboratories throughout the country, both commercial and college, have co-operated with the Committee in the making of extensive series of tests of mortar and concrete, many of them involving the manufacture and breaking of a large number of 8 x 16 in. cylinders, made under various conditions and tested at different ages. The names of the laboratories co-operating are given in the report of Committee C-9 to the American Society for Testing Materials at the last annual meeting of the Society. Reference also may be made to this report for further details.

The work which is now being carried on by the various sub-committees, in addition to the research on impurities, includes the investigation of the best methods of making and testing concrete specimens; the laws of mixtures of different sized aggregates; methods of sampling and testing field concrete; determination of the relative values of various strength tests with mortars; and methods of testing for voids, weights, density, specific gravity and consistency.

Many of these tests are in line with, and in fact a continuation of the various series carried on by your Institute committee and presented in the report of this committee in 1914. Attention is called to the very valuable results relating to sizes and shapes of specimens, both in strength, consistency and storage, which are presented in that report and printed in the *Journal*, October-November, 1914.

Since the presentation of last year's report, certain of the tests have been continued. One of the most notable series is that showing the remarkable influence of method of storage of concrete specimens upon their strength. The results of these tests made at the University of Illinois and originally printed in the 1914 report of your committee, have been extended up to two years and the results are shown in the table below. It is to be noted that the specimens stored in dry air show very low strength in comparison with all of the others. This comparatively low strength of concrete in dry storage

indicates the necessity for further study of reinforced concrete in building construction, where the building is closed in and subjected to heat at early ages. Fortunately, in practice a concrete structure nearly always is exposed to the elements for a number of weeks after the concrete is placed, so that there is opportunity for it to attain fair strength before being shut in.

COMPARATIVE STRENGTH OF CONCRETE AT DIFFERENT AGES AND CONSISTENCIES STORED UNDER DIFFERENT CONDITIONS.

Each Value is an Average of four 6-in. Cylinders. Proportions 1 : 2 : 4 by weight. Normal Consistency. Tests at University of Illinois.

Consistency.	Water in Mixing.	Storage.	Compressive Strength at Different Ages, lb. per sq. in.							
			7 Days.	14 Days.	21 Days.	28 Days.	2 Mo.	6 Mo.	1 Yr.	2 Yr.
Dry.......	8.4	Damp sand............	1751	2140	2658	2615	3056	3941	3700	4890
Normal...	9.3	Damp sand............	1390	1775	1816	1820	3063	3431	3768	4042
Wet.......	10.2	Damp sand............	1103	1354	1623	1657	2410	3281	3760	3914
Normal...	9.3	Air....................	1481	2061	2126	2116	2232	2049	2350	2189
Normal...	9.3	Coated with paraffine. Damp sand..........	....	....	....	2314	2521	3339	3675	4235
Normal...	9.3	Damp sand*...........	....	....	....	....	2734	3433	3945	....
Normal...	9.3	Air*..................	....	....	....	....	2208	1888	2000	....

* Made from dry stone; for all other test pieces the stone had been thoroughly wet before mixing.

Respectfully submitted,

SANFORD E. THOMPSON, *Chairman*,
CLOYD M. CHAPMAN,
A. T. GOLDBECK,
RUSSELL S. GREENMAN,
NATHAN C. JOHNSON,
WILLIAM M. KINNEY,
ARTHUR N. TALBOT.

DISCUSSION.

Mr. Bates. MR. BATES.—I note that the author of the paper referred to various methods of storage of test pieces during their aging. I wondered whether he used the German method of storage, which, you recall, consists of one day in a damp closet, six days in water and twenty-one days in air. There are apparently several reasons which makes such storage very advantageous and desirable. First, it simulates somewhat the actual conditions under which concrete itself ages. There is a decided attempt to keep concrete very moist and to prevent drying out as much as possible during the early periods. Secondly, in view of the fact that the hydration of cement is colloidal, it follows directly that a too wet test specimen will have a lower strength than one somewhat drier. As the colloid absorbs water from a very dry condition it gains strength. When, however, the colloid has in it more than a certain amount of water the strength again begins to fall off.

It would appear from the results which the Germans obtained that their ratio of time of water and air storage produced such a drying out that there is just the proper amount of water present to give the higher strengths. In a paper which I read at Atlantic City last year on the "Finer Grinding and Higher SO_3 Content in Cement," I used this method of storage for the 28-day specimens, and the same ratio between the water storage and the air storage for the longer periods. The result has been that in some cement mortars we obtained from two to three times the strength obtained when the specimens were stored constantly in water.

Mr. Wig. MR. WIG.—I would like to point out that the rate of drying is very much different for small cubes and concrete in a column 18 or 20 in. in diameter which will affect the strength differently.

Mr. Chapman. MR. CHAPMAN.—Just a word in reply to Mr. Wig. These tests show the effect of dry storage on specimens, not concrete in use. We do not interpret them that way. This is an investigation to determine the effect on the strength of specimens of different methods of storage. To add to what Mr. Bates has said, the committee are investigating many other methods of storage, subdividing wet and dry storage in different ratios. These are not reported in this particular series of tests.

THE USE OF THE UNIVERSAL SAND TESTER.

By Cloyd Chapman.*

In this age of wonderful progress along practical lines we are very prone to think that anything that is new to us as individuals is new to the world. We are very liable to assume when we hear, for example, of a movement looking toward the finer grinding of cement, that it is a new movement, something our grandfathers knew nothing about, when as a matter of fact the movement began almost with the introduction of Portland cement. Fifty years ago John Grant, member of the Institute of Civil Engineers and one of the greatest pioneers in the use of Portland Cement, presented to the Institute tables of results of tests on the effect of fine grinding and showed the greatly increased strength of 1:1, 1:2 and 1:3 mortars due to the use of finer cement. It is interesting to note, however, that John Grant used 36 and 50 mesh sieves in separating out his fine cement for tests and that he states that as the cement came from the manufacturer it contained about 20 per cent which would remain on a 36-mesh sieve and about 30 per cent on a 50-mesh sieve.

While it may be just a little disappointing to us that the idea of fine grinding did not originate in this generation, we can at least have the satisfaction of knowing that we have gone further with it than our grandfathers went. From the 36 and 50-mesh sieves we have advanced to the 200-mesh sieve and have reached the time when even this sieve is not considered fine enough for the purpose.

And so it is with sand testing. We seem to have the idea that nobody before our time ever thought that the sand used had anything to do with the quality of the concrete produced. True it is, that all too little attention was given to sand and very little effort made to get and use the best, but that statement applies with equal force to our own day and practice. We may know better, but we don't do much better.

But the early users of Portland cement recognized the effect of the quality of the sand on the resulting product. When the Southern High-level Sewer was built in London, the first sewer on the south side of the River Thames, it was built of Portland cement and the preliminary tests made included tests of the sands. Grant describes tests made in 1862 and 1863 of three available sands described as "clean Thames sand," "clean pit sand" and "loamy pit sand." His results are shown in Table I, taken from his paper "On the Strength of Portland Cement."

Note that Grant tried proportions ranging from 1:1 up to 1:5 so as to arrive at the proper proportions to use to get the required strength. He did not simply test one proportion, such as 1:3, and attempt to decide on any such limited information. This work was done over fifty years ago. How much have we improved on the idea since then?

* Engineer of Tests, Westinghouse, Church, Kerr & Co.

All sand to be used in important concrete work, and most concrete work is important, should be tested. Sand is an extremely variable commodity. It has a value to the concrete maker directly proportional to its quality. It is not at all uncommon to find that of two sands available for a particular job one will give as high compressive strength in a 1:3 mixture with cement as the other will give in a 1:2 mixture with the same cement. Assuming that the strength which the poorer sand would give in a 1 : 2 : 4 mixture is

TABLE I.—THE RESULTS OF 960 EXPERIMENTS WITH PORTLAND CEMENT, WEIGHING 112 LB. TO THE IMPERIAL BUSHEL, GAGED NEAT, AND WITH DIFFERENT PROPORTIONS OF VARIOUS KINDS OF SAND, SHOWING THE BREAKING WEIGHT IN POUNDS ON A SECTIONAL AREA OF 2.25 SQ. IN. 1862 AND 1863.

Age and Time in Water.	1 Week.	1 Month.	3 Months.	6 Months.	9 Months.	12 Months.
Neat cement	445.0	679.9	877.9	978.7	995.9	1075.7
Clean Thames sand:						
1 to 1	97.0	309.3	367.0	546.8	607.8	700.3
1 to 2	52.5	123.5	254.5	425.1	431.5	458.3
1 to 3	27.0	58.0	135.5	232.4	....	320.6
1 to 4	....	32.5	109.0	157.0	....	221.6
1 to 5	....	21.0	88.5	95.5	....	122.3
Clean pit sand:						
1 to 1	152.0	326.5	549.6	639.2	718.7	795.9
1 to 2	64.5	166.5	451.9	497.9	594.4	607.5
1 to 3	44.5	91.5	305.3	304.0	383.6	424.4
1 to 4	22.0	71.5	153.0	275.6	....	317.6
1 to 5	....	49.0	123.5	218.8	....	215.6
Loamy pit sand:						
1 to 1	114.2	274.7	448.3	586.5	600.1	645.5
1 to 2	53.0	130.5	354.0	415.6	516.8	533.2
1 to 3	21.0	68.0	149.0	274.2	321.3	358.4
1 to 4	....	60.5	118.5	225.5	226.7	244.4
1 to 5	....	31.5	78.5	141.0	154.3	166.2

sufficient, and assuming that all other factors are constant, such as price of cement, coarse aggregate, etc., the better sand is worth a premium equal to the price of 0.45 bbl. of cement for every yard of concrete installed. Or, viewed from the quality instead of the money standpoint, the better sand used in the same proportions will give something like 30 to 50 per cent stronger concrete. In the face of these facts a very small proportion of the sand used for concrete is properly tested.

Natural sands differ from one another in many particulars. Their many qualities may be classified under four primary heads. These are:

(*a*) Kind of rock, *i. e.*, mineral composition.

(*b*) Angularity or smoothness of grains.

(*c*) Size of grains.

(*d*) Presence of organic or foreign matter, or cement coatings.

The other qualities of sands are dependent upon one or more of these four leading characteristics. For instance, specific gravity depends upon the

kind of rock, voids depend chiefly upon the form and size of grains, and weight depends upon the kind of rock, shape of grain, and size of grains, and so on.

The quality of a sand as regards the concrete it will make is dependent upon all four, but chiefly upon the kind of rock, size of grains, and foreign matter, although their relative importance may not be in the order named. Angularity or smoothness of grain probably has much less effect on the quality of concrete produced than the other qualities named.

Sands from widely different localities may differ greatly in all of the four prime qualities, depending upon the character of the country rock and the manner in which nature formed the sand bank.

On the other hand, sands from the same locality are not so liable to vary in all of these four prominent respects. That is to say, the kind of rock of which the sand is composed is not liable to differ greatly in sands from the same geological deposit; the angularity or smoothness of the grains is not very likely to be radically different in sands deposited in the same manner by the same agency; the presence of organic matter, cementing materials or coating is very liable to be reasonably uniform in any particular bank of sand, if surface stripping is properly attended to. The one main quality of sand which does vary widely in almost every deposit is the size of the grains.

Almost all sand banks are stratified. The various strata are composed of sands of varying degrees of coarsness, some very fine, some very coarse, and some medium. In one end of the sand pit the fine-sand strata may greatly predominate and in the other end the coarse. In other words, the chief variation in the quality of sand coming from a given deposit is not in the kind of rock composing it, or in the outward shape of the grains, or in the amount of organic matter or cementing matter it contains, but in its relative proportion of different sizes of grains. This is the one chief variable.

The quality of concrete which any sand will make depends in varying degree upon all the qualities of the sand. The effect of some of the qualities is unknown and indeterminate, because the various qualities are so dependent upon each other that they cannot be segregated and evaluated.

It is, therefore, impossible to predict with any certainty from the results of any determinations of the various physical and chemical qualities of a sand what grade of concrete it will make. The one reliable means of ascertaining the quality of concrete that a given proportion of a particular sand, coarse aggregate and Portland cement will produce, is to make the concrete and test the result. There is no short cut; if you want to know you must find out by trial. But having "found out," that is, having made the necessary tests, the details of which we will not discuss at this time, and having determined the quality of concrete that the materials under consideration will produce when mixed in the proportions which will be used on the job, there still remains the necessity for keeping check on the quality of the materials actually delivered to and used on the job, if predetermined results are to be even approximated.

Would you permit an engineer or an architect who had charge of a large and important concrete job for you to test a sample of the cement at the

beginning of the job and then never test any of the cement actually used on the job? Why not? That procedure is followed with sand, and sand is much more variable than cement. Poor sand has been the cause of ten times as much poor concrete as has poor cement. It is very much safer to omit testing the cement than the sand.

Testing sand is so easily done that there remains no excuse for neglecting the testing of sand as it is delivered to the job. Once the quality of a sand from a particular source has been established by proper investigation, there remains but one variable that must be checked up continuously and that variable is the size of grains.

Any method which will determine quickly and with reasonable accuracy whether the granularmetric analyses of the sand shipments received agree with the analysis of the sample tested when the suitability of the sand was determined, will serve to check the quality of the sand as it is used on the job.

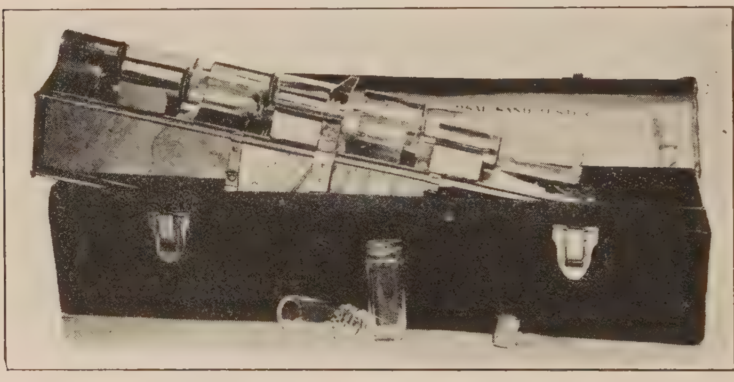

FIG. 1.—SAND TESTER WITH CASE

Several methods of accomplishing this end suggest themselves to you. But those methods are either crude and inaccurate, or they do not give a record of the result, or they are cumbersome and slow, or require an expert operator to carry them out.

Every method except one with which the author is familiar or has heard proposed, is open to one or more of these objections. The one method which seems to solve the problem and to provide a quick, easy, accurate record of the size analysis of sand without requiring an expert operator or laboratory equipment is by means of a sand tester recently put on the market for this particular purpose.

In its essential features this tester is nothing but a series of screens and a series of glass receptacles in which the volume of the sand which is retained on each sieve is measured. But as constructed it accomplishes much more than this and in its operation it is much simpler than even this simple description would imply.

The instrument and its carrying case are shown in Fig. 1. It consists of a box of sheet copper in which are soldered a series of wire screens or sieves placed at an angle of about 45 deg. with the narrow sides of the box. Fig. 2 shows one of these instruments constructed with glass sides to show the internal arrangement of screens and to show the action of the sand and water when in operation.

There are five of these screens which divide the box into six compartments. Each of these compartments opens into a chamber attached to one side of the box and into each of these sides chambers is screwed a glass cylindrical measuring receptacle. The chambers are staggered along the side of the box so that the glass receptacle attached to each chamber lies beside the next adjacent chamber. The openings between the main box and the side chambers are the full size of the chamber. The glass receptacles seat against rubber washers to make a water-tight joint. One end of the main box is provided with a round opening and a rubber plug for closing the opening.

On one side of the box are two rods on which slide a platen holding a record sheet and also an index finger. Both the platen and the index finger

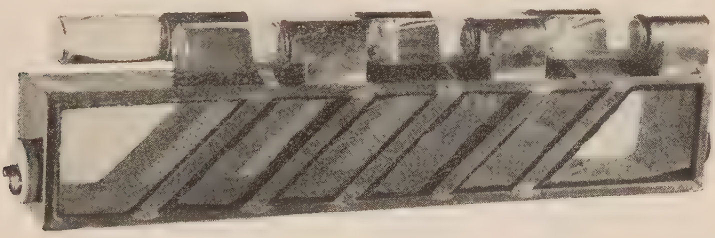

FIG. 2.—SAND TESTER WITH GLASS SIDES

slide freely along the rods the full length of the box and both are easily removable by springing the slide clips off the rods. There are no other parts to the instrument except a cylindrical metal cup for measuring the sample to be tested.

To make a sand test with this instrument the small measure is filled from the sand pile and emptied into the box. Water is added until the box is nearly full. The rubber stopper is inserted in the end of the box and, holding the instrument in an inclined position with the glass receptacles uppermost, the sand is washed down through the sieves by shaking the instrument in all directions. The glass receptacles, being inverted, do not fill with water and therefore act as air bells or cushions for the impulsing water while the sand is being shaken down through the screens.

This instrument is next turned slowly over about its longitudinal axis until the glass receptacles are on the under side, and the sand, now separated into six sizes, is washed from the compartments of the box into the chambers. The tester is then turned on end, with the receptacles upright, and the sand washed out of the chambers into the six glass receptacles.

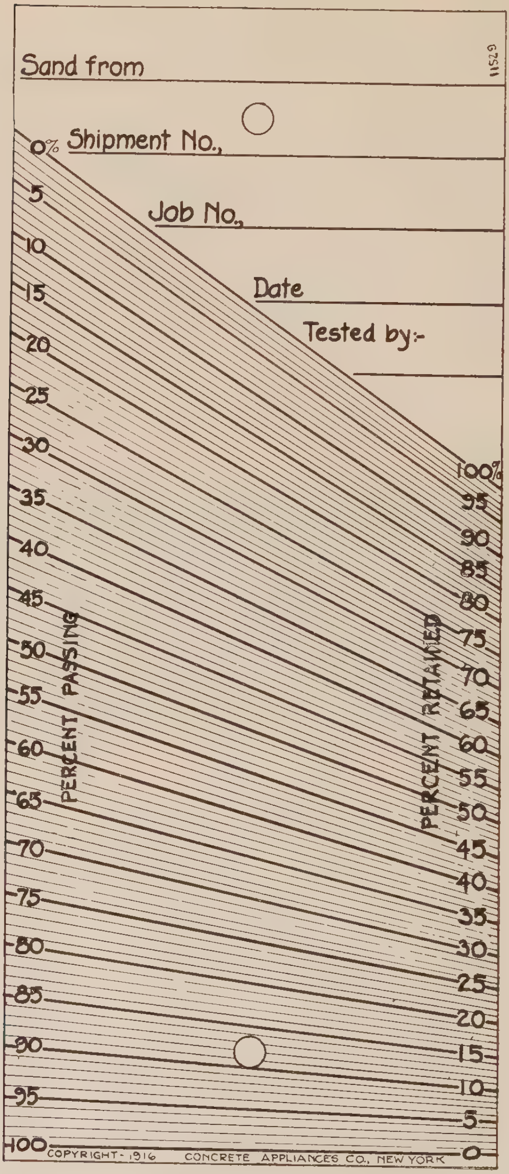

FIG. 3.—SAND TEST RECORD SHEET

It is to be noted that by means of the water in the instrument, not only is sieving more effectually accomplished, but also on absolute moisture condition, that of saturation, is established.

The depth of the sand in each receptacle is therefore proportional to the quantity of each size of sand. The platen carrying the record sheet is now clipped on the slide rods and the index finger is snapped into position on the same rods. The form of record sheet generally used is shown in Fig. 3. It provides blank spaces for the registration of the necessary data to identify the sample and provides a variable length scale of one hundred parts on which percentage of the total sample may be read directly even through the total volume of the sample may vary between rather wide limits, as is explained later.

The bottom line of the record sheet is placed opposite the bottom of the sand in the lowest receptacle and the index finger is moved along until it is in line with the top of the sand in the bottom receptacle. A line is then drawn with a pencil across the record sheet, using the lower edge of the index as a straight edge. The index is then moved up until it is opposite the bottom of the sand in the next receptacle. The platen is moved up until the pencil line just drawn coincides with the lower edge of the index. The index is then moved up to the top of the sand in the second receptacle and another line drawn across the record sheet along the lower edge of the index. By repeating this procedure, there are produced on the record sheet a series of horizontal lines whose distances apart are equal to the depths of the sand in the six receptacles. The top line will cut the top diagonal line of the record sheet at some point depending upon the size and grading of the sample. At this point of intersection, representing 100 per cent, or all of the sand, a vertical line is drawn on the record sheet. This vertical line will intersect all the horizontal lines previously drawn. At these points of intersection may be read to the right the percentage retained on, and to the left the percentage passing, the various screens in the box.

Fig. 4 shows a record sheet with the horizontal and vertical lines drawn and makes plain how readily the percentages are read off from the record.

In order that the operator may know at a glance whether the sand he has tested is acceptable or not, it is convenient to have a standard diagram prepared for the sand under inspection. This standard diagram may be prepared for each grade of sand by making several preliminary tests on samples of the sand known to be satisfactory. These results are averaged and a record slip marked to indicate the allowable range in percentage of the size of particles. Such a record slip marked for a particular sand and allowing a 2 per cent, variation from the mean, or total variation of 4 per cent, is shown in Fig. 4. With such a diagram in his possession, an inspector can tell at once whether the sand tested varies materially from the standard analysis established for that particular sand.

Now that this tester and its operation has been described, one of the first questions to be asked is, how closely do results thus obtained by wet screening and volume measuring of the various sizes compare with the usual method of dry screening and weighing of the different sizes? While it is not

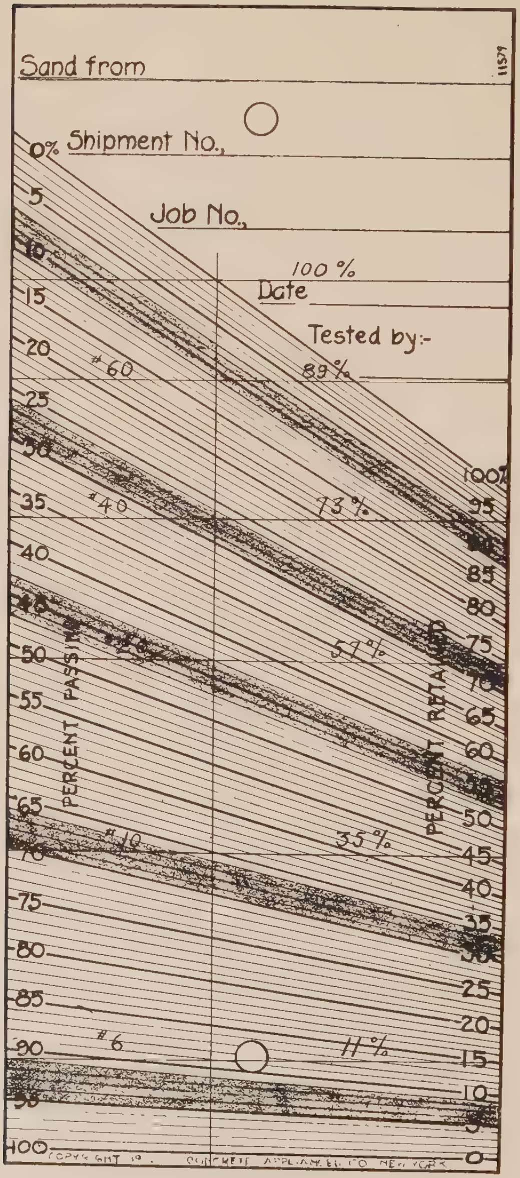

FIG. 4.—SAND TEST RECORD.

essential that the two methods shall agree, they yet correspond within close limits with a greater efficiency of separation of the fine sizes by the instrument.

To compare these results a series of curves are shown in Fig. 5, in which the curves shown in broken lines were obtained with the dry method and the curves shown in full lines and bearing the same number were obtained with the tester.

One very noticable result that is most strongly in the favor of the wet screening method is the fact that a greater quantity of fine grains are passed through the small sieves when screened wet than when screened dry. This is

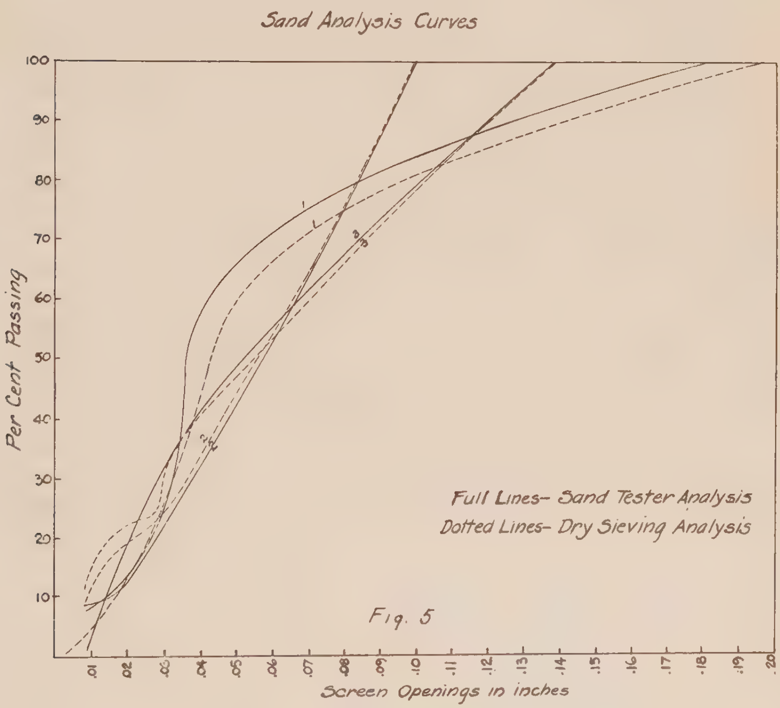

FIG. 5.

probably due to the washing action and the separation of fine particles adhering to other particles. As the fine particles are the ones which have by far the greatest influence upon the strength of the concrete, it is of the greatest importance that their proportion be correctly determined. One of the sands used in the tests represented by the curves in Fig. 5 was a clean dredged sand, one of the best sands found around New York City, yet when dried for screening there was enough of the fine material adhering to the other

particles to give deceptive results. This is a typical illustration of the superiority of wet screening over dry screening.

As to the time required to make a test with the instrument, about ten minutes on the average is consumed in taking the sample, making the separation, drawing the record diagram, and cleaning out the instrument for another test. As compared with any other known method, this is very rapid.

In conclusion, it should be emphasized that sand, as well as other materials of construction, should be regularly inspected prior to, not after, its use; that this inspection must include a determination of the granular-metric composition of the sand; and that there is now available a quick, simple, reliable and accurate means for accomplishing this result.

REPORT OF COMMITTEE ON BUILDING BLOCKS AND CEMENT PRODUCTS.

This committee considered carefully the existing specifications and subjects which come under its jurisdiction, and deems it advisable to make several changes and additions to former specifications and reports as follows:

1. We find that municipalities which regulate the use of concrete blocks and concrete architectural stone usually combine in one requirement our Standard Nos. 11 and 12.

2. We find that these standards can be simplified and improved if combined.

3. We therefore offer a substitute for Standards Nos. 11 and 12, entitled the "Proposed Standard Specifications and Building Regulations for the Manufacture and Use of Concrete Architectural Stone, Building Blocks and Brick."

4. We recommend that this Association through its executive officers, or some committee appointed for the purpose, communicate with the building departments of all our larger cities to the end that they adopt this standard in full.

5. We offer proposed specifications for the manufacture and use of concrete drain tile and sewer pipe.

Although these specifications are not by any means the last word on the subjects, we urge their adoption and use until better ones are offered, because of the urgent need of standards that can be used by engineers throughout the country. Up to the present time they have had no common standard to refer to.

The adoption of the proposed specifications on concrete drain tile will make Standard No. 9 obsolete.

Respectfully submitted,

ROBERT F. HAVLICK, *Chairman.*
D. A. ABRAMS.

PROPOSED STANDARD SPECIFICATIONS AND BUILDING REGULATIONS FOR THE MANUFACTURE AND USE OF CONCRETE ARCHITECTURAL STONE BUILDING BLOCKS AND BRICK.

(As proposed by the Committee on Building Blocks and Cement Products, February 14, 1916.)

1. Concrete architectural stone and building blocks for solid or hollow walls and concrete brick made in accordance with the following specification and meeting the requirements thereof may be used in building construction.

2. *Tests.*—Concrete architectural stone, building blocks for hollow and solid walls and concrete brick must be subjected to

(*a*) Compression and (*b*) absorption tests.

All tests must be made in a testing laboratory of recognized standing.

3. *Ultimate Compressive Strength.*—(*a*) Solid concrete stone, building blocks and brick.

In the case of solid concrete stone, blocks and brick the ultimate compressive strength at 28 days must average fifteen hundred (1500) lb. per sq. in. of gross cross-sectional area of the stone as used in the wall and must not fall below one thousand (1000) lb. per sq. in. in any case.

(*b*) Hollow and two piece building blocks.

The ultimate compressive strength of hollow and two piece building blocks at 28 days must average one thousand (1000) lb. per sq. in. of gross cross-sectional area of the block as used in the wall, and must not fall below seven hundred (700) lb. per sq. in. in any case.

4. *Gross Cross-Sectional Areas.*—(*a*) Solid concrete stone, blocks and brick.

The gross cross-sectional area shall be considered as the minimum area in compression.

(*b*) Hollow building blocks.

In the case of hollow building blocks, the gross cross-sectional area shall be considered as the product of the length by the width of the block. No allowance shall be made for the air space of the block.

(*c*) Two piece building blocks.

In the case of two piece building blocks, if only one block is tested at a time, the gross cross-sectional area shall be regarded as the product of the length of the block by one-half of the width of the wall for which the block is intended. If two blocks are tested together, then the gross cross-sectional area shall be regarded as the product of the length of the block by the full width of the wall for which the block is intended.

5. *Absorption.*—The absorption at 28 days (being the weight of the water absorbed) must not exceed ten (10) per cent of the weight of the dry sample when tested as hereinafter specified.

6. *Samples.*—At least six samples must be provided for the purpose of testing. Such samples must represent the ordinary commercial product and shall be selected from stock. In cases where the material is made and used in special shapes and forms too large for testing in the ordinary machine, smaller specimens shall be used as may be directed. Whenever possible, the tests shall be made on full size samples.

7. *Compression Tests.*—Compression tests shall be made as follows:

The sample to be tested must be carefully measured and then bedded in plaster of paris or other cementitious material in order to secure uniform bearing in the testing machine. It shall then be loaded to failure. The compressive strength in pounds per square inch of gross cross-sectional area shall be regarded as the quotient obtained by dividing the total applied load in pounds by the gross cross-sectional area, which area shall be expressed in square inches computed according to Article 4. When such tests must be made on portions of blocks, both pieces of the block must first be carefully measured and if these blocks are for hollow walls, the amount of air space must be carefully calculated. The samples shall then be bedded to secure uniform bearing, and loaded to failure. In this case, however, the compressive strength in pounds per square inch of net area must be obtained and the net area shall be regarded as the minimum bearing area in compression. The average of the compressive strength of the two portions of block shall be regarded as the compressive strength of the samples submitted. This net compressive strength shall then be reduced to compressive strength in pounds per square inch of gross cross-sectional area as follows:

The net area of a full size block shall be carefully calculated and the total compressive strength of the block will be obtained by multiplying this area by the net compressive strength obtained above. This total compressive strength shall be divided by the gross cross-sectional area as figured by Article 4 to obtain the compressive strength in pounds per square inch of gross cross-sectional area.

When testing other than rectangular blocks, great care must be taken to apply the load at the center of gravity of the specimen.

8. *Absorption Tests.*—The sample shall be first thoroughly dried to a constant weight at a temperature not to exceed 212° F., and the weight recorded. After drying, the sample shall be immersed in clean water for a period of forty-eight hours. The sample shall then be removed; the surface water wiped off, and the sample re-weighed. The percentage of absorption shall be regarded as the weight of the water absorbed divided by the weight of the dry sample multiplied by one hundred (100).

9. *Limit of Loading.*—(*a*) Hollow walls of concrete building blocks.

The load on any hollow walls of concrete blocks, including the superimposed weight of the wall, shall not exceed one hundred and sixty-seven (167) lb. per sq. in. of gross area. If the floor loads are carried on girders or joists resting on cement block pilasters filled in place with slush concrete mixed in the proportions of one (1) part cement, not to exceed two (2) parts of sand and four (4) parts of gravel or crushed stone, said pilasters may be loaded not to exceed three hundred (300) lb. per sq. in. of gross cross-sectional area.

(*b*) Solid walls of concrete blocks.

Solid walls built of architectural stone, blocks or brick and laid in Portland cement mortar shall not be loaded to exceed three hundred (300) lb. per sq. in. of gross cross-sectional area.

10. *Girders and Joists.*—Wherever girders or joists rest upon walls in such manner as to cause a concentrated load of over four thousand (4000) lb. the blocks supporting the girders or joists must be made solid for at least eight (8) in. from the inside face of the wall, except where a suitable bearing plate is provided to distribute the load over a sufficient area to reduce the stress so it will conform to the requirements of Article 9.

When the combined live and dead floor loads exceed sixty (60) lb. per sq. ft., the floor joists shall rest on a steel plate not less than three-eights ($\frac{3}{8}$) in. thick and of a width one-half to one (1) in. less than the wall thickness. In lieu of said steel plate the joists may rest on a solid block which may be three (3) or four (4) in. less in wall thickness than the building wall, except in instances where the wall is eight (8) in. thick, in which case the solid blocks shall be the same thickness as the building wall.

11. *Thickness of Walls.*—(*a*) The thickness of bearing walls shall be such as will conform to the limit of loading given in Article 9. In no instance shall bearing walls be less than eight (8) in. thick. Hollow walls eight (8) in. thick shall not be over sixteen (16) ft. high for one story or more than a total of twenty-four (24) ft. for two stories.

(*b*) Walls of residences and buildings commonly known as apartment buildings not exceeding four stories in height, in which the dead load does not exceed sixty (60) lb. or the live load sixty (60) lb. per sq. ft., shall have a minimum thickness in inches as shown in Table I.

TABLE I.

No. of Stories.	Basement.	First Story.	Second Story.	Third Story.	Fourth Story.
1	8	8	..	..	..
2	10	8	8	..	..
3	12	10	8	8	..
4	16	12	10	8	8

12. *Variation in Thickness of Walls.*—(*a*) Wherever walls are decreased in thickness the top course of the thicker wall shall afford a solid bearing for the webs or walls of the course of concrete block above.

13. *Bonding and Bearing Walls.*—Where the face only is of hollow concrete blocks, and the backing is of brick, the facing of hollow blocks must be bonded to the brick, either with headers projecting four (4) in. into the brick work, every fourth course being a header course, or with approved ties, no brick backing to be less than eight (8) in. thick. Where the walls are made entirely of concrete blocks, but where said blocks have not the same width as the wall, every fifth course shall overlap the course below by not

less than four (4) in. unless the wall system alternates the cross bond through the wall in each course.

14. *Curtain Walls.*—For curtain walls the limit of loading shall be the same as given in Article 9. In no instance shall curtain walls be less than eight (8) in. in thickness.

15. *Party Walls.*—Walls of hollow concrete blocks used in the construction of party walls shall be filled in place with concrete in the proportion and manner described in Article 9.

16. *Partition Walls.*—Hollow partition walls of concrete blocks may be of the same thickness as required in hollow tile, terra cotta, or plaster blocks for like purposes.

PROPOSED SPECIFICATIONS FOR MANUFACTURE OF CONCRETE SEWER PIPE BY MACHINE.

(As Proposed by the Committee on Building Blocks and Cement Products, February 14, 1916.)

1. *Portland Cement.*—The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials, adopted August 16, 1909, as revised to date by said Society.

2. *Fine Aggregate.*—Fine aggregate shall consist of sand, crushed stone or gravel screenings, graded from fine to coarse, and passing when dry a screen having four (4) square meshes per linear in.; shall be preferably of silicious material, clean, coarse, free from dust, soft particles, loam, vegetable or other deleterious matter; not more than thirty (30) per cent shall pass a sieve having fifty (50) meshes per linear in. and not more than six (6) per cent shall pass a sieve having one hundred (100) meshes per linear in. Fine aggregates shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate by weight shall show a strength equal to or greater than the strength of 1 : 3 mortar of the same consistency made with the same cement and standard Ottawa sand.

In no case shall fine aggregate contain more than three (3) per cent by weight of clay, loam or other organic, deleterious or soluble matter.

3. *Coarse Aggregate.*—Coarse aggregate shall consist of inert material, as pebbles or crushed stone graded in size, retained on a screen having four (4) meshes per linear in.; shall be clean, hard and durable; shall contain no vegetable or other deleterious matter, and shall be free from dust, soft, flat or elongated particles. The greatest dimension of coarse aggregate shall not be greater than one-half the wall thickness of the sewer pipe in which it is used.

4. *Water.*—Water shall be clean, free from oil, acids, strong alkalis, vegetable matter or other deleterious material.

5. *Measuring Materials.*—A bag of Portland cement (94 lb. net) shall be considered one (1) cu. ft. The method of measuring the materials for the concrete shall be one which will insure separate and uniform proportions of each of the materials at all times.

When cement in bulk is used, the cement shall be accurately measured in such manner as to insure that the measured amount of cement used as the equivalent of one bag, in the proportions as specified below, shall be ninety-four (94) lb.

6. *Mixing.*—The concrete materials shall be mixed in a machine mixer that will produce concrete uniform in color and homogeneous in appearance.

7. *Retempering.*—Retempering of mortar or concrete which has partially hardened, that is, remixing with additional materials or with water, shall not be permitted.

8. *Proportions.*—For sewer pipe up to and including ten (10) in. in diameter, the concrete shall be mixed in the proportions of one (1) bag of Portland cement to not more than two and a half ($2\frac{1}{2}$) cu. ft. of fine aggregate. For sewer pipe over ten (10) in. in diameter, in which coarse aggregate having the maximum size of particles of one-half ($\frac{1}{2}$) in. or more is used, the concrete shall be mixed in the proportions of one (1) bag of Portland cement to not more than four (4) cu. ft. of fine and coarse aggregate measured separately and in no case shall the mixture contain more than two and a half ($2\frac{1}{2}$) cu. ft. of fine aggregate to each bag of cement.

Methods of proportioning the various ingredients shall be used which will secure uniform measurements at all times.

9. *Consistency.*—Concrete shall be mixed as wet as can be used and permit the immediate removal of the outer casings from the sewer pipe. The pipe shall show on the outer surface web-like markings, or water-marks, indicating free moisture after the removal of the jackets. Interior surfaces of the sewer pipe shall show trowel marks caused by free water coming to the surface under the troweling action of the revolving packer-head or core, in case such is used.

10. *Forming.*—Sewer pipe shall be made in such a manner as to insure a dense and uniformly compacted product with smooth ends and inner surfaces. Pipe shall be formed so as to prevent laminations or planes of weakness. Bell-end pipe shall be manufactured by a method which will insure continuity of the pipe and bell-end of each pipe shall be of cylindrical section, the size being designated by the interior diameter. The thickness of the pipe shall be practically uniform throughout and shall not be less than one-tenth ($\frac{1}{10}$) the diameter for sizes up to ten (10) in., nor one-twelfth ($\frac{1}{12}$) the diameter for larger sizes with a minimum thickness of three-fourths ($\frac{3}{4}$) of an inch. The diameter shall not vary more than three (3) per cent from that specified.

11. *General Clauses.*—Immediately after removal of the casings, pipe shall be placed in a curing chamber not exposed to sun and wind, and may be hardened either by the application of water vapor, which is known as the steam curing method, or by sprinkling, known as the water curing method.

Steam curing. Within one (1) hour after removing the outer casings, the pipe shall be placed in a closed chamber where the temperature is not lower than 50° F. and protected from currents of air and from all exposure which will tend to cause evaporation of moisture from the concrete. Within twelve (12) hours after removing the pipe from the machine, the temperature in the chamber shall be raised to between 100° and 160° F., and an atmosphere saturated with water vapor introduced through a perforated pipe laid throughout the length of the kiln as near the floor as possible. When the outside temperature during the day does not fall below 50° F., the pipe shall be cured as above described for not less than forty-eight (48) hours, after which they may be removed and piled in the yard. They shall then be sprinkled not less than three (3) times daily for seven (7) days. When the temperature of the outside atmosphere falls below 50° F., pipe shall be cured for seventy-

two (72) hours, after which they may be piled in the yard but need not receive further treatment.

Water Curing. Whenever it is found impracticable to introduce steam into the chamber, the pipe shall be placed in a closed chamber and protected from currents of air and from all exposure, which will tend to cause evaporation of moisture from the concrete. After they have sufficiently hardened so that the application of water does not injure them, they shall be kept constantly wet on the surface by sprinkling with water for not less than seven (7) days whenever the temperature does not fall below 50° F., and for fourteen (14) days when the temperature does fall below 50° F. After removal from the chamber, the pipe shall be stored in the yard.

12. *Time of Storage.*—Pipe shall not be shipped until they have been stored in the yard for not less than fourteen (14) days after steaming or sprinkling.

13. *Selection of Test Specimens.*—One specimen shall be selected, if desired, out of any one hundred (100) pipe furnished. If any single pipe so selected shall fail to meet the requirements specified or shall fail to pass every test to which it is subjected, two additional specimens shall be selected at random and submitted to the prescribed tests. If these specimens pass the tests, the lot shall be accepted, but if they fail, the entire lot from which they were selected shall be rejected.

14. *Testing.*—In making the strength tests of sewer pipe the methods shall conform to those in use by the city for which the pipe is made.

SPECIFICATIONS FOR MANUFACTURE OF CONCRETE DRAIN TILE.

(As Proposed by the Committee on Building Blocks and Cement Products, February 14, 1916.)

1. *Portland Cement.*—The cement shall meet the requirements of the Standard Specifications for Portland Cement of the American Society for Testing Materials, adopted August 16, 1909, as revised to date by said Society.

2. *Fine Aggregate.*—Fine aggregate shall consist of sand, crushed stone or gravel screenings, graded from fine to coarse, and passing when dry a screen having four (4) square meshes per linear in.; shall be preferably of silicious material, clean, coarse, free from dust, soft particles, loam, vegetable or other deleterious matter; not more than thirty (30) per cent shall pass a sieve having fifty (50) meshes per linear in. and not more than six (6) per cent shall pass a sieve having one hundred (100) meshes per linear in. Fine aggregate shall be of such quality that mortar composed of one (1) part Portland cement and three (3) parts fine aggregate by weight shall show a strength equal to or greater than the strength of 1 : 3 mortar of the same consistency made with the same cement and standard Ottawa sand.

In no case shall fine aggregate contain more than three (3) per cent by weight of clay, loam or other organic, deleterious or soluble matter.

3. *Coarse Aggregate.*—Coarse aggregate shall consist of inert material, as pebbles or crushed stone graded in size, retained on a screen having four (4) meshes per linear in.; shall be clean, hard and durable; shall contain no vegetable or other deleterious matter, and shall be free from dust, soft, flat or elongated particles. The greatest dimension of coarse aggregate shall not be greater than one-half the wall thickness of the drain tile in which it is used.

4. *Water.*—Water shall be clean, free from oil, acids, strong alkalies, vegetable matter or other deleterious material.

5. *Measuring Materials.*—A bag of Portland cement (94 lb. net) shall be considered one (1) cu. ft. The method of measuring the materials for the concrete shall be one which will insure separate and uniform proportions of each of the materials at all times.

When cement in bulk is used, the cement shall be accurately measured in such manner as to insure that the measured amount of cement used as the equivalent of one bag, in the proportions as specified below, shall be ninety-four (94) lb.

5. *Mixing.*—The concrete materials shall be mixed in a machine mixer that will produce concrete uniform in color and homogeneous in appearance.

7. *Retempering.*—Retempering of mortar or concrete which has partially hardened, that is, remixing with additional materials or with water, shall not be permitted.

8. *Proportions.*—For drain tile up to and including ten (10) in. in diameter, the concrete shall be mixed in the proportions of one (1) bag of Portland cement to not more than three (3) cu. ft. of fine aggregate. For the manufacture of drain tile over ten (10) in. in diameter, in which coarse aggregate having the maximum size of particles of one-half ($\frac{1}{2}$) in. or more is used, the concrete should be mixed in the proportions of one (1) bag of Portland cement to not more than five (5) cu. ft. of fine and coarse aggregate measured separately and in no case shall the mixture contain more than three (3) cu. ft. of fine aggregate to bag of cement.

Methods of proportioning the various ingredients shall be used which will secure uniform measurements at all times.

9. *Consistency.*—Concrete shall be mixed as wet as can be used and permit the immediate removal of the outer casings from the drain tile. The tile shall show on the outer surface web-like markings, or water-marks, indicating free moisture after the removal of the jackets. Interior surfaces of the drain tile shall show trowel marks caused by free water coming to the surface under the troweling action of the revolving packer-head or core, in case such is used.

10. *Forming.*—Drain tile shall be made in such a manner as to insure a dense and uniformly compacted product with smooth ends and inner surfaces. Drain tile shall be formed so as to prevent laminations or planes of weakness. Each tile shall be of cylindrical section, the size being designated by the interior diameter. The thickness of the tile shall be practically uniform throughout and shall not be less than one-tenth ($\frac{1}{10}$) the diameter for sizes up to ten (10) in., nor one-twelfth ($\frac{1}{12}$) the diameter for larger sizes with a minimum thickness of one-half ($\frac{1}{2}$) in. The diameter shall not vary more than three (3) per cent from that specified.

11. *General Clauses.*—Immediately after removal of the casings tile shall be placed in a curing chamber not exposed to sun and wind, and may be hardened either by the application of water vapor, which is known as the steam-curing method, or by sprinkling, known as the water-curing method.

Steam Curing. Within one (1) hour after removing the outer casings, the tile shall be placed in a closed chamber where the temperature is not lower than 50° F. and protected from currents of air and from all exposure which will tend to cause evaporation of moisture from the concrete. Within twelve (12) hours after removing the tile from the machine, the temperature in the chamber shall be raised to between 100° and 160° F., and an atmosphere saturated with water vapor introduced through a perforated pipe laid throughout the length of the kiln as near the floor as possible. When the outside temperature during the day does not fall below 50° F., the tile shall be cured as above described for not less than forty-eight (48) hours, after which they may be removed and piled in the yard. They shall then be sprinkled not less than three times daily for seven (7) days. When the temperature of the outside atmosphere falls below 50° F., tile shall be cured for seventy-two (72) hours, after which they may be piled in the yard but need not receive further treatment.

Water Curing. Whenever it is found impracticable to introduce steam

into the chamber, the tile shall be placed in a closed chamber, and protected from currents of air and from all exposure, which will tend to cause evaporation of moisture from the concrete. After they have sufficiently hardened so that the application of water does not injure them, they shall be kept constantly wet on the surface by sprinkling with water for not less than seven (7) days whenever the temperature does not fall below 50° F., and for fourteen (14) days when the temperature does fall below 50° F. After removal from the chamber the tile shall be stored in the yard.

12. *Time of Storage.*—Tile shall not be shipped until they have been stored in the yard for not less than fourteen (14) days after steaming or sprinkling.

13. *Selection of Test Specimens.*—One specimen shall be selected, if desired out of any one hundred (100) tile furnished. If any single tile so selected shall fail to meet the requirements as hereinafter specified or shall fail to pass every test to which it is subjected, two additional specimens shall be selected at random and submitted to the prescribed tests. If these specimens pass the tests, the lot shall be selected, but if they fail, the entire lot from which they were selected shall be rejected.

14. *Testing.*—In making strength tests of drain tile the methods proposed by Standard Specifications for drain tile of the American Society for Testing Materials shall be used and the specimens shall have the minimum average ordinary supporting strengths in accordance with the following tables:

"(*a*) Class No. 1 B.—No. 1 B tile are intended to be suitable for supporting the load in the worst material in a trench having a grade line 5 ft. deep. They shall have minimum average ordinary supporting strengths (calculated as specified in "Strength Tests of Drain Tile," Section 8) in accordance with the following table:

REQUIRED AVERAGE ORDINARY SUPPORTING STRENGTH FOR CLASS No. 1 B TILE.

DIAMETER OF TILE, IN.	LB. PER LINEAR FOOT.
10	600
12	700
14	800
16	900
18	1000
20	1100
22	1100
24	1200

"(*b*) Class No. 1 A.—No. 1 A shall be made of good materials by the most approved method, and are intended to be suitable for supporting the load in the worst material in a trench having a grade line 7 ft. deep.

"The inner surface of the tile shall be free from defects. The outer surface shall be free from broken blisters, lumps or flakes which are thicker than

20 per cent of the thickness of the tile, or whose diameter is greater than 15 per cent of the inner diameter of the tile, and such defects as are allowed shall not be of such nature as to appreciably weaken the tile when laid in the ditch.

"The tile shall have minimum average ordinary supporting strengths (calculated as specified in 'Strength Tests of Drain Tile,' Section 8) in accordance with the following table:

REQUIRED AVERAGE ORDINARY SUPPORTING STRENGTH FOR CLASS No. 1 A TILE.

Diameter of Tile, In.	Lb. per Linear Foot.
12	900
14	1000
16	1200
18	1300
20	1400
22	1550
24	1700
26	1800
28	1900
30	2000
32	2050
34	2150
36	2250

"Class No. 1 Extra A.—No. 1 Extra A tile shall be extra good, and are intended to be suitable for supporting the load in the worst material in a trench having a grade line 10 ft. deep. They shall be either vitrified, salt-glazed, clay tile, or thoroughly seasoned concrete tile, made of the best materials, by the most approved method.

"The inner surface of the tile shall be free from defects. The outer surface shall be free from broken blisters, lumps or flakes, which are thicker than 16 per cent of the thickness of the tile, or whose diameter is greater than 12 per cent of the inner diameter of the tile, and such defects as are allowed shall not appreciably weaken the tile when laid in the ditch.

"The tile shall have minimum average ordinary supporting strength (calculated as specified in 'Strength Tests of Drain Tile,' Section 8) in accordance with the following table:

REQUIRED AVERAGE ORDINARY SUPPORTING STRENGTH FOR CLASS No. 1 EXTRA A TILE.

Diameter of Tile, In.	Lb. per Linear Foot.
12	1000
14	1200
16	1500

Diameter of Tile, In.	Lb. per Linear Foot.
18	1700
20	2100
22	2300
24	2500
26	2600
28	2800
30	3000
32	3200
34	3300
36	3500

Tile not meeting the above specifications shall be rejected.

DISCUSSION.

Mr. Wig. MR. WIG.—There are a number of clauses in the proposed specifications that are not clear. For example, tile are required to be of uniform thickness throughout. This would cause the rejection of small tile which are corrugated. Another clause refers to "clay, loam and other vegetable matter;" clay and loam are not vegetable matter. Tests are required to be made at the center of gravity of the specimen. The intent is apparently that the load be applied directly over the center of gravity. There does not appear to be any absorption test for drain tile. There is no statement of the manner of making the strength test. As these tests are not made to determine the general quality but only the strength of the tile, and since the quality of small tile is very important in some sections of the country, particularly the West, I think the specifications should provide more complete tests for determining quality of small tile.

Mr. Chapman. MR. CHAPMAN.—The supporting capacity is of less importance in the case of small tile than of large sizes. Durability is affected by other properties than ability to carry a load. Density is more important than strength for many purposes. To insure satisfactory quality it is necessary to specify density for all sizes. Absorption as an indicator of density should be an essential test for all drain tile.

Another point to which attention should be drawn is the incompatibility of specifying the mechanical analysis of fine aggregates and also a certain strength as compared with that obtained with Ottawa sand. If strength or density or both are desired they should be specified, but not the granularmetric analysis of the aggregate.

Prof. Abrams. PROF. ABRAMS.—The explanation of some of the phrases in the specifications for drain tile is that Committee C-6 of the American Society for Testing Materials has prepared specifications for drain tile which will be submitted to that society at its next meeting. That committee has been studying the subject for several years and our committee thought that it should not recommend the results of the work of that committee. Since these results will not be made public until later, hence our committee has recommended for temporary use the present proposed specifications of the American Society for Testing Materials. Before the next meeting of the Institute our committee can probably submit a more satisfactory specification.

I am ready to agree with Mr. Chapman that a granulometric specification such as we have submitted may exclude sand satisfactory in some work, but many sands may give the same strength as standard Ottawa sand and yet be unsatisfactory in actual practice. All we can do is to try to make our requirements represent our knowledge of the subject as closely as possible.

(It was voted to receive and print the report and to continue the committee.)

DURABILITY OF CONCRETE PIPE.

By J. H. Libberton.*

When those who planned the American Concrete Pipe Association saw their hopes crystalized into the organization now bearing that name, they conceived for such activities a broad field—one which would cover drainage and sewerage so far as the use of concrete pipe was concerned. So, in using the term concrete pipe as a title in the following brief discussion, it is intended to include not only concrete sewer pipe but concrete drain tile as well.

Many authorities, whose opinions are based on experience far greater than the writer can claim, have already discussed in a technical way the merits of concrete sewer pipe and the effect of ground water or sewer acids upon concrete. This paper does not intend to touch upon that subject further than to analyze some of the causes of general opinions which exist as to the durability of concrete pipe and to quote from several city engineers whose experience with concrete pipe has been definite and whose statements are authoritative.

Directed by commercial self-interest or influenced by other viewpoints, there are many who claim that concrete pipe is not a suitable material for use in drains or sewers. As a result, many have arisen to theorize and argue for or against concrete pipe. Such persons have usually aimed to support their contentions by the findings of laboratory experiments of one kind or another and their deductions have necessarily been largely theoretical. Nevertheless, the fact remains that there are hundreds of miles of concrete pipe in satisfactory operation both in sewer and land drainage service, in many cases extending over periods up to thirty-five or forty years.

Communications received from city engineers in practically every known location where concrete has been used indicate satisfaction with the product. Evidently theory disagrees with fact, and when such is the case, it is rather better to accept fact and revise theory.

No doubt some are fixed in the opinion that concrete pipe will not give satisfactory service, but such impressions are largely the result of adverse advertisements or literature which has dwelt especially on concrete failures: literature which has usually emanated from competitive interests. It is not the intention here to discuss the ethics of advertising, but rather to call attention to the possible reasons why we should expect other than a favorable opinion generally existing regarding the service of concrete pipe.

Concrete is no different from any other construction or building material. To render effective service it must be used after a manner meeting with established requirements which have been formulated as a result of experience. Any building or construction material is likely to prove a dismal failure unless handled by those who know how to secure from its use the best results.

* Division Engineer, Information Bureau, Universal Portland Cement Co.

Concrete pipe in the vast majority of cases has given exceptional service in the ground, but, like other materials, there have been a few failures, insignificant, however, if compared with the great total mileage which is giving satisfaction. If competitive interests choose to advertise only the failures of concrete pipe, disregarding entirely the successes, the public mind is, of course, influenced to believe that all concrete pipe must be a failure. If, on the other hand, only successes are advertised, concrete pipe is assumed to be giving general satisfaction. After all, then, the opinion which is generally held of concrete pipe is entirely dependent upon the nature of its advertising.

Probably no better analysis has been made of the present condition than that of the city engineer of Spokane, Morton Macartney, who sums up the situation in the following paragraph: "I believe the prejudice against cement pipe is largely due to a campaign carried on by the manufacturers of other kinds of pipe largely through the showing up of defective cement pipe when such defects were due to poor workmanship rather than inherent defects in the class of material used."

So far, those who represent the collective concrete pipe interests have never maintained an active organization to promote favor and advertise the merits of their product. The only advertising which has been generally distributed, if such matter may be called advertising, is that which has resulted in anything but a favorable attitude toward concrete pipe. Is it any wonder, then, that many base their opinion entirely upon the advertisement of concrete pipe failures?

No product, whether made from wood, steel, clay or concrete, is independent of materials or workmanship. Concrete pipe is equally dependent upon the materials and methods used in its manufacture, but unlike most other products, it is hidden away under the surface of the soil where no one knows of or can appreciate the efficient service it is rendering. If satisfactory, no one ever hears of it; if the reverse is true, competitive interests welcome and take advantage of the opportunity to spread broadcast its failure.

One might, for the purpose of illustration, liken concrete pipe to an office clerk whose services are entirely above reproach for a considerable period. One day a mistake comes up for attention. Until then, the employer hardly knew that this clerk was working for him. After a couple of errors, he is ready to discharge him, forgetting entirely the days of efficient service. This is particularly true if the clerk be a newcomer. Employees old in the service can make one mistake after another and still remain because their long connection with the company has become an asset in their favor. The charitable employer, however, will always take into consideraton the fact that mistakes, if corrected, furnish the opportunity for improvement and development, and appreciate that under proper guidance his clerk will no doubt develop into a more efficient employe than some of the older heads who have become rusty in service.

So with concrete pipe, which is in a sense somewhat of a newcomer into the drainage and sewerage field. The public is the employer. If carelessness in manufacture causes concrete pipe to appear for censorship, it should be remembered that errors, corrected and eliminated, insure the quality of the

product and prevent a recurrence of the same condition. To carry our comparison further, the public's attention should be called to the hundreds of miles of pipe which have been giving satisfactory service for years and which make the few failures look insignificant in comparison.

If only *one* good concrete pipe has been manufactured, millions more can be made after the same process; so, rather than theorize on the action of sewage acids and ground water on concrete pipe, would it not be better to analyze critically the good pipe which has been made?

An analysis of the essentials which influence the quality of concrete pipe discloses the following:

Materials: Cement, sand, gravel or stone and water.
Proportions of ingredients.
Method of forming.
Method of hardening or curing.

Low absorption also might be called one of the indications of good pipe, but this is largely dependent upon the essentials above mentioned.

There is no need to detail good concrete practice in pipe manufacture. It is the same whether forming pipe or placing a retaining wall. Good materials are necessary; sufficient water and cement are required to insure maximum density; the method of forming should be such as to thoroughly compact the concrete; and the hardening or curing should be carried on in such a way that the concrete may have opportunity to reach its maximum strength under conditions which will not rob the newly-molded product of any of the moisture which was originally placed in the mixture. Pipe so produced will withstand satisfactorily exposure to any of the soil acids or "alkalis" about which so much concern centers.

Lean and porous concrete will be disintegrated by the action of plain water alone; the writer has seen many such cases. Concrete tile have been taken from neutral ground, the lower half of which was entirely washed away and the upper half considerably disintegrated. This action results not only from erosion but from solvent action as well, the water easily breaking up the porous concrete. This statement is supported by laboratory experiments also, where the cement has easily been dissolved out of porous specimens of drain tile.

The experiments of actual practice and those of the laboratory of the Bureau of Standards indicate that the long-discussed action of "alkali" is not one of solution but rather principally intensive crystallization of chemical salts in the porous walls of the pipe. Subsequent disintegration is largely a mechanical action and not a dissolving one as sometimes claimed. Proof of this statement can be had by an examination of the structure of the disintegrated tile which always shows a soft interior with a hard inner and outer shell. If the action were a dissolving one, the disintegration would begin at the shell. As it is, evidence indicates that crystallization has taken place on the interior, which in turn causes the pipe wall to swell and weaken. With concrete pipe of maximum density, such a condition could and does not occur because, being no opportunity for penetration, there would be no crystallization of alkali in the interior of the wall structure.

The best possible conditions have often been provided for the disintegration of concrete tile not only from alkali and soil acids but from pure water as well, on account of the former tendency in tile manufacture to make a wall as porous as possible in the belief that thus drainage would be more efficiently accomplished. With this in mind, not only was a minimum of water used, but a minimum of cement also, with the result that the pipe was of an extremely dry consistency and of a proportion often as lean as 1 part of cement to 9 parts of fine sand. Many manufacturers sold their products by means of an exibition which consisted of blowing tobacco smoke through the tile wall. The writer has often witnessed this demonstration, as well as the facility with which water would run through the tile wall. Naturally such concrete could not long resist any erosive or solvent action, and fast made room for a better product. Fortunately, today there are more manufacturers of concrete drain pipe using a 1 : 3 than a 1 : 5 mixture; and practically all are a unit on the necessity for low absorption and high strength.

Rudolph Hering makes an interesting statement regarding the relation between disintegration and quality.

"I have seen some sewage tanks in Europe, particularly those in Hampton, south of London, in which the sewage had very much disintegrated the concrete and cement of which the tanks were built. The sewage at Hampton is much stronger than the average sewage in the United States. I noticed that the original concrete did not seem to be very dense; therefore, the sewage has been absorbed by capillarity and caused decomposition. There were patches on the walls of the tank which showed no disintegration; their surfaces were invariably smooth and showed no great porosity. They were discolored but not soft."

Mr. Hering thus strikes at the seat of the entire discussion—that of quality. There is yet no record where properly made, cured and proportioned concrete pipe have disintegrated either in the drain or in the sewer.

The work of Committee C-6 of the American Society for Testing Materials has done much toward standardizing processes and methods of tests; and with the completion of its work will have established an accurate basis upon which to judge all kinds and qualities of drain tile, and we may hope its activities may be later extended to sewer pipe as well. If the specifications of this committee are passed, none but first-class pipe will be allowed.

Reinforced concrete, on account of low cost, has for some years been practically above competition in the construction of large sewers. Engineers generally have admitted that properly mixed and proportioned concrete, used in reinforced monolithic concrete sewer construction, is satisfactory for the purpose. Concrete pipe sewers can be placed just as satisfactorily as reinforced monolithic concrete sewers, and there are many arguments in favor of the former, on account of the ease of inspection, not only while manufactured, but before placed in the ground. The principal problem has been one of getting the manufacturer to appreciate his responsibility and to keep him from assuming the attitude that after his product is covered up his responsibility ends.

The concrete pipe has, however, assumed considerable responsibility itself, as will be indicated by addressing a letter to city engineers who have had experience with concrete in sewer construction, both in the form of pipe and when placed as a monolith.

It might be well to quote a few expressions from such engineers in support of this statement:

CITY OF WILMINGTON, DELAWARE: "We have used approximately 10,000 ft. of premolded reinforced concrete pipe varying in size from 24 to 72 in. The first laid eight years ago has been in constant service and has given perfect satisfaction. Similar results have been realized with the remainder.

(Signed) "ALEXANDER J. TAYLOR, *Chief Engineer.*"

SPRINGFIELD, MASSACHUSETTS: "This cement pipe was purchased by the city of a local manufacturer, and owing to the fact that it was bought under no particular specifications or inspection, it proved to be more or less defective, especially the pipe bought during the latter part of our use of the same. About ten years ago we began to use Portland cement concrete for the construction of all sewers in this city greater than 2 ft. in diameter. This use of concrete has been most satisfactory and has resulted in a large saving to the city over and above the cost of sewers formerly constructed of brick.

"I may add that all sewers at the present time are laid by the city on the day's labor plan. We are gradually replacing the old cement pipe sewers laid many years ago with vitrified clay as rapidly as serious defects appear. However, we are firm believers here in Portland cement concrete, and I have no doubt that a very superior pipe sewer can be constructed with this material, with or without proper steel reinforcement. We may further add that all our larger concrete sewers have been laid with plain monolithic concrete and without reinforcement.

(Signed) "CHARLES M. SLOCUM, *Deputy City Engineer.*"

SALT LAKE CITY, UTAH: "The first concrete sewer was laid in this city in 1886, 36 to 48 in. size and about 7 miles in length. This sewer shows no perceptible deterioration and is in excellent condition. Several years ago an intercepting sewer of reinforced concrete of 30 to 40-in. diameter was built, which has been quite leaky, but this has been due entirely to poor construction. The machine-made concrete pipe has proven to be very satisfactory. Prices here run about the same as vitrified pipe. Compression tests show the concrete pipe to be considerably stronger than the vitrified. It is very dense and smooth. At the end of this season the city will have approximately 75 miles of concrete pipe sewers in operation.

(Signed) "SYLVESTER Q. CANNON, *City Engineer.*"

CITY OF MILWAUKEE, WISCONSIN: "We have approximately 487 miles of sewer in this city, 130 miles of which is 30 to 120 in. in diameter. During the period from January 1, 1908, to January 1, 1915, the Sewer Department has re-laid a total of 2700 lineal ft., of which there was 851 lineal ft. or vitrified

pipe sewer, 1070 lineal ft. of cement pipe sewer and 779 lineal ft. of which we have no record as to kind of pipe. This period of seven years is typical of the percentage of pipe re-laid in the past twenty years. We have used concrete pipe for over twenty-five years. Machinery used in this city for the past four years for manufacturing a wet-mix concrete pipe in sizes up to 18 in. and reinforced concrete pipe 24 to 54 in., has given, in our opinion, splendid results.

(Signed) "ED. F. LEIDEL, *Chief Engineer*."

CITY OF JACKSON, MICHIGAN: "So far as this department is aware, the concrete pipe sewers have been satisfactory and perform their function efficiently. The accompanying total will indicate the extent which concrete pipe sewers have been constructed in this city. (The total indicates that $210,538 have been spent for concrete, while the total for all sewer construction in the city has been $489,764, or 43 per cent of the expenditures for sewers have been for concrete.)

(Signed) "H. S. MCGU, *Sanitary Engineer*."

GREEN BAY, WISCONSIN: "Concrete has been the material used in building the sewers larger than 24 in. for the last seven years, with the exception of a small amount of brick and vitrified clay sewers. Concrete has given entire satisfaction in every respect and we are thoroughly pleased with it.

(Signed) "AUGUST BRAUNS, *City Engineer*."

RICHMOND, INDIANA: "We have used concrete for sewer construction for a number of years and in general have obtained good results. Of course, you know that concrete can be made good, bad or indifferent, and this applies especially to sewer work. With good materials, honest workmanship and careful inspection, there is no doubt, in my opinion, that concrete sewers will give a full dollar's worth of service for every dollar of cost.

(Signed) "FREDERIC R. CHARLES, *City Civil Engineer*."

ALBANY, N. Y.: "We have used reinforced concrete pipe varying in size from 24 to 60 in. inside diameter for an intercepting sewer and for trunk sewers with much satisfaction. We originally used the reinforced concrete pipe for the intercepting sewer because its use did not make it necessary to clutter up this street with piles of sand, stone, cement, concrete mixers, etc. We have since used it for trunk sewers because it has been more economical than monolithic concrete."

(Signed) "FRANK R. LANAGAN, *City Engineer*."

JANESVILLE, WISCONSIN: "In our sanitary sewer system the main outlet sewer extending from the junction of the east and west side systems at the end of the syphon under Rock River to the discharge below the city, a distance of about $1\frac{3}{4}$ miles, is of 48-in concrete pipe construction. There is also some 36-in. pipe and about $\frac{1}{3}$ of a mile of 24-in. trunk line connections of concrete

pipe. This work has been in use since 1908 and to date has proven perfectly satisfactory without repair or attention of any kind.

(Signed) "C. V. KERCH, *City Engineer*."

SAVANNAH, GEORGIA: "Properly constructed reinforced concrete pipe is especially adaptable to supersaturated soil. I am of the opinion that it stands every necessary test. I do not believe in the use of vitrified clay pipe above a diameter of 24 in. We have had practically no trouble in the construction of the concrete pipe sewers. They have been laid in very bad bottom. There is practically no seepage in the joint and I do not hesitate in saying that I am a firm believer in the use of this type of material. It should be properly constructed, thoroughly investigated during the construction and carefully laid.

(Signed) "E. R. CONANT, *Chief Engineer*."

SALT LAKE CITY, UTAH: "Machine-made cement pipe is being manufactured in Salt Lake City and is being used as lateral sewers, house connections and other drainage purposes, and it is first-class and in many respects superior to the vitrified pipe which is manufactured here. I witnessed a test on a sample of 8-in pipe taken at random from the stock and it withstood 60 lb. pressure. The manner in which this pipe is made renders it perfectly smooth on the inner surface and all joints are uniform. The many letters and reports that I have read from people who have condemned the use of cement pipe will certainly need to change their opinion when their attention is drawn to the machine-made cement pipe as this pipe is now made.

(Signed) "G. F. McGONAGLE, *City Engineer*."

HALIFAX, NOVA SCOTIA: "We have been laying concrete sewers for about a quarter of a century. We have never had a failure in this work, though over twenty years ago there was a good deal of public criticism and an investigation was held. With ordinary sewage I feel certain there is no disadvantage in using concrete. Up to the time concrete was adopted, brick sewers were constructed for the larger sizes and as the unions forced the rate of wages up, we were obliged to adopt concrete construction exclusively and have used no other material in the last twenty-five years. There are certain acids used in dye work and other industries which, if not sufficiently diluted with water, will affect the concrete, but if the discharge from such buildings empties into a considerable volume of ordinary sewage, I do not think it would have any effect upon the concrete. We have been able to make the concrete sewers as cheap as the smaller sizes of clay pipe and cheaper than the larger sizes. As concrete sewer pipe is now made by machinery, it is considerably cheaper than clay pipe, and being made much more compactly than formerly, is not surpassed by any other material."

(Signed) "F. W. W. DOANE, *City Engineer*."

To these we might briefly summarize the experience of Max Gary, of Berlin, who, in 1907, secured by means of letters to city engineers and

others some interesting information in answer to the question, "How long have you been using cement pipe for drainage purposes?"

The replies, tabulated, are as follows:

29	places	have	used	concrete	pipe	from	1 to 5	years.
46	"	"	"	"	"	"	5 to 10	"
44	"	"	"	"	"	"	10 to 15	"
24	"	"	"	"	"	"	15 to 20	"
20	"	"	"	"	"	"	20 to 25	"
21	"	"	"	"	"	"	25 to 40	"

If we accept the words of the large number of engineers who have used concrete pipe with satisfaction, why need we discuss the composition of sewage and its effect upon concrete pipe? If, however, there is a demand for technical discussion, let us admit for the moment that theoretically sewage has a disintegrating action on concrete. Practice has shown that such action rarely, if ever, takes place. Then the logical line of reasoning must lead us to the question: Why is concrete pipe not affected by sewage or ground water? We can come only to the following conclusions: First, that concrete pipe is generally of such quality as to withstand any tendency toward being affected by disintegrating influences which are ordinarily met with in a sewer or drain.

Second, that if raw sewage experimentally does have a disintegrating effect on average concrete, the actual conditions in the sewer must be such that the pipe surface is protected with slime, or else the sewage is so diluted as to have no action.

Third, that the isolated examples cited of disintegration are traceable to poor concrete, due to the use of poor material, improper methods of manufacture or to careless methods of curing.

Fourth, that if concrete pipe, as it has been manufactured during the last thirty or more years, have given such exceptional satisfaction, as the experience of city and drainage engineers indicates, it is reasonable for us to assume that the concrete pipe as manufactured today from standard Portland cement and clean, well-graded aggregates, combined with standard methods of manufacture and curing, will vie with the Pyramids in its endurance?

THE CHEMISTRY OF PORTLAND CEMENT.

By G. A. Rankin.*

Portland cement is now amongst the most valuable of manufactured products, its aggregate value being probably only second to that of iron and steel. Forty years ago its use was limited and it was manufactured on only a small scale; at the present time its use is so widespread that the annual production in this country alone is about 100,000,000 barrels. The chief use of Portland cement is as a substitute for stone. In some respects, concrete is superior to stone as a building material but under many conditions it is not so durable as the best building stone. There is reason to believe, however, that it may be possible to produce a cement which will yield a concrete of much greater durability than the Portland cement now made does. Indeed, as the demand for Portland cement has increased and as the requirements of engineers have called for material of better quality, the manufacturers have been able to meet these demands and to improve continuously the quality of their product. Now, it is to be noted that this continuous improvement has been brought about almost entirely by improvements in the mechanical appliances and methods of the industry, and owes very little to new ideas of how to make Portland cement based on a knowledge of what the constitution of the product really is. And in view of this fact that such progress has been possible under such circumstances, it would seem not unreasonable to look forward to further improvement in cement now that its constitution has been definitely ascertained.

The following pages present the results of an investigation of the constitution of Portland cement clinker and of the cementing value of the several constituents; these results enable us to indicate the nature and direction of the research work now required to ascertain the composition and best mode of production, of the cementing material best adapted to any particular purpose.

Portland cement clinker is the result of chemical combinations of the three oxides, lime, alumina, silica; but beside these three—which are the essential components—two others, namely, magnesia and ferric oxide, always occur to some extent in commercial cement. The average of a large number of chemical analyses of American-made Portland cement is, according to Meade,

CaO	62.0 per cent	Fe_2O_3	2.5 per cent
Al_2O_3	7.5 per cent	MgO	2.5 per cent
SiO_2	22.0 per cent	SO_3	1.5 per cent

From this it is evident that more than 90 per cent of an average Portland cement consists of the three oxides, CaO, Al_2O_3, SiO_2; one would expect, therefore, that its properties are due mainly to the presence of the above three components and that the relatively small admixture of the other oxides

* Geophysical Laboratory, Carnegie Institution of Washington, Washington, D. C.

exerts at most a wholly secondary influence. Indeed, it has been shown that good Portland cement can be made from the three pure oxides, lime, alumina and silica, in the proper proportions.

Now, ordinary chemical methods enable us to ascertain the aggregate proportion of each oxide present, but they yield us no information as to the manner in which these oxides are combined with one another—in other words, as to the substances which actually are present in the clinker and are responsible for its characteristic properties. The determination of this question is very important, for this reason; that, until we know what these substances actually are, we cannot hope to improve the method of making Portland cement, or to improve the quality in any desired direction, except by cut-and-try methods; and it is generally recognized that such empirical methods are much less certain, and take a vastly longer time to reach the goal, than methods based on a real knowledge of the factors in the problem. The determination of this question has been the object of a very large number of investigations; but the experimental basis of most of this work has been altogether insufficient to decide the several questions at issue. There has been, in general, a failure to realize the fact that a system so complicated as this can be solved only by proceeding systematically, using as a guide the principle known as the phrase rule and establishing definite criteria for the recognition of the several substances which occur.

Cement clinker is a mixture of substances of very similar properties, and is, moreover, exceedingly fine grained, as a consequence of which it is a matter of some difficulty to make quantitative determination of the constituents; but this difficulty can be surmounted by studying separately each of the presumable constituents of the clinker and determining definite values of certain properties which serve to characterize it and to distinguish it from other possible constituents. Accordingly the first problem is to isolate and determine all the possible compounds of lime, alumina and silica which we may expect to find in Portland cement clinker, to establish their relations at high temperatures and to ascertain their optical characteristics which constitute the most convenient and satisfactory criterion of the identity of the several substances.

These characteristic properties of the several solid substances, containing only CaO, Al_2O_3, SiO_2, which are likely to occur in Portland cement have been determined at the Geophysical Laboratory of the Carnegie Institution of Washington in the course of a systematic investigation of all compounds formed when any mixture of these oxides is heated to a high temperature. In American-made Portland cements the relative proportions of these oxides vary only between comparatively narrow limits: CaO—60 to 64; Al_2O_3—5 to 9; SiO_2—19 to 25; in other words, in considering this special problem we have to deal with a very restricted portion of the field of the whole system CaO-Al_2O_3-SiO_2.

THE TERNARY SYSTEM CaO-Al_2O_3-SiO_2.

In order to work out this system completely it proved necessary to investigate about 1000 different mixtures of these three oxides and to make

about 7000 heat treatments and microscopical examinations of the resultant products. Each such mixture, which was always made up of especially pure materials, was alternately fused and ground to a fine powder, the fusions being made in a platinum crucible to avoid contamination, in order to obtain a thoroughly combined product. Each of these products was heated in an electric furnace, the temperature of which was carefully controlled and measured, until all changes had ceased, when it was quickly chilled; and the resultant material was subjected to a complete optical study. This procedure, which was carried out systematically, enables one to determine the crystalline phases present at temperatures ranging from that at which melting begins to that at which the charge is completely melted; and thus to ascertain the melting temperature and optical properties of all compounds of lime, alumina, silica which form when any mixture of these three oxides is heated.

The data thus obtained can be interpreted most readily if they are plotted in three dimensions; the concentration (composition) of each mixture is represented by a point within an equilateral triangle* on the horizontal plane, the magnitude of the corresponding temperature by the distance above this plane. It would lead us too far to go into details of the construction and properties of such a model;† suffice it to say that the series of surfaces thus described represent the melting temperatures of all products obtained when any mixture of the three oxides is heated progressively to higher and higher temperature. A photograph of such a model is reproduced in Fig. 1. As can be seen from the photograph, the model resembles a relief map of a mountainous region; each mountain peak is the melting point of a pure component or of a pure compound; the mountain slopes represent the melting temperatures of a component or compound in ternary mixtures; the points where the rivers in the valleys meet to form a lake are the lowest melting temperatures, known as eutectic points. This model, when interpreted with the aid of the principles underlying such equilibria, enables one to specify the order in which the several crystalline substances will form when any mixture composed entirely of lime, alumina and silica is heated, and also to state what are the final products when the reactions have gone to completion.

Let us consider the crystalline substances which will form when a mixture composed only of these three oxides in the proportions such that they will produce a good Portland cement, is heated. For this purpose a diagram such as Fig. 2 is useful. This diagram is a projection on the horizontal plane of that portion of the solid model necessary for our present purpose. The corresponding temperatures are here represented by isothermal lines, which are completely analogous to the contour lines on an ordinary map. In this

* In such a diagram, the pure components, CaO, Al_2O_3, SiO_2, are represented by the apices of the triangle; the binary systems, CaO-Al_2O_3, CaO-SiO_2 and Al_2O_3-SiO_2, by the sides of the triangle, and ternary mixtures by points within the triangle.

† Those interested in the details of all this work are referred to previous papers, particularly Rankin and Wright, *Am. J. Sci.* (4) 39 (1915), 1–79, and G. A. Rankin, J. *Ind. Eng. Chem.* 7, 466 (1915).

diagram the group of dots represents mixtures of CaO, Al_2O_3, SiO_2 from which Portland cement of good quality can be made.

This group of dots, it will be noted, are all included within the triangular area formed by lines connecting the compositions of the three compounds, tricalcium silicate, dicalcium silicate and tricalcium aluminate. For that reason, if any mixture of CaO, Al_2O_3 and SiO_2, such as is represented by one

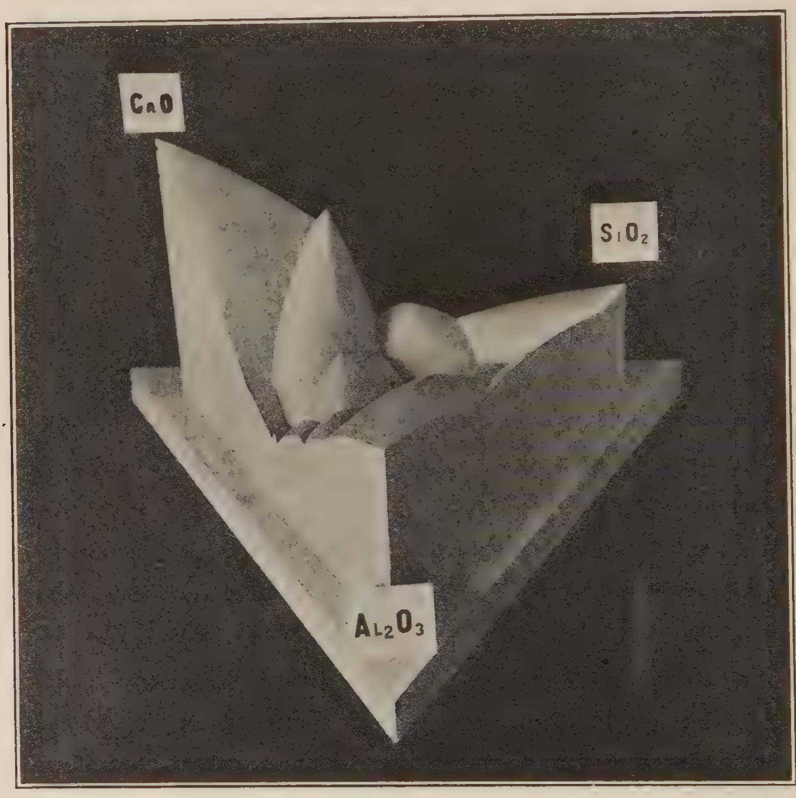

FIG. 1.—SOLID MODEL OF CONCENTRATION-TEMPERATURE DIAGRAM OF THE TERNARY SYSTEM, $CaO-Al_2O_3-SiO_2$.

of these dots, is heated so that all chemical reactions are completed, the final product obtained will be made up of the above three compounds.

The constituents of Portland cement clinker made up only of the oxides CaO, Al_2O_3, SiO_2 are, therefore, the three compounds $3CaO.SiO_2$, $2CaO.SiO_2$ and $3CaO.Al_2O_3$. Each of these compounds has optical properties peculiar to itself which serve to distinguish it from the rest. The several characteristic optical and crystallographical properties were obtained by a study

of each compound by itself. These values are constants for the individual compounds in all mixtures made up from pure CaO, Al_2O_3 and SiO_2; i. e., the final products resulting when such mixtures are heated, are present as individuals of constant optical properties and not as solid solutions.

Microscopical examination of commercial Portland cement clinker shows it to be made up largely (over 90 per cent) of these three compounds $2CaO.SiO_2$, $3CaO.SiO_2$ and $3CaO.Al_2O_3$. It would appear, therefore, that the value of Portland cement as a cementing material when mixed with water is largely due to one or more of these compounds. Before taking up the cementing value of each of these compounds, however, let us consider their formation when Portland cement is burned.

For this purpose let us follow the reactions which take place when a mixture whose composition is CaO (as $CaCO_3$) 68.4 per cent, Al_2O_3 8.0 per cent, SiO_2 23.6 per cent (point *P*, Fig. 2), is slowly heated. This mixture, made up only of the pure oxides, lime, alumina, silica, when properly burned, will produce a good Portland cement. When such a mixture is heated, the first change is the evolution of the CO_2; the lime then unites with the other components to form the compounds $5CaO.3Al_2O_3$ and $2CaO.SiO_2$ (both of which form readily) probably in the order named, since the former has a lower melting point than the latter; subsequently these two compounds unite in part with more lime and the compounds $3CaO.SiO_2$ and $3CaO.Al_2O_3$ appear. This formation of the last two compounds—a process which goes on very slowly in mixtures of their own composition—is materially facilitated by the circumstance that in the ternary mixtures a portion of the charge has already melted and promotes reaction by acting as a flux or solvent. The temperature at which this flux first appears is 1335° C., the eutectic temperature for the three compounds $2CaO.SiO_2$, $5CaO.3Al_2O_3$, $3CaO.Al_2O_3$. As the temperature of burning gradually rises above 1335° C., the relative amount of flux increases and the rate of formation of $3CaO.Al_2O_3$ and $3CaO.SiO_2$ increases correspondingly. At a temperature somewhat above 1335° C. the compound $5CaO.3Al_2O_3$ will have completely melted in the flux and the formation of $3CaO.Al_2O_3$ is complete. The substances present as crystals at this stage are $3CaO.SiO_2$, $3CaO.Al_2O_3$, $2CaO.SiO_2$ and free CaO. Of these the $3CaO.SiO_2$ is rapidly *increasing* in amount, due to combination of $2CaO.SiO_2$ with CaO, while the amounts of solid $2CaO.SiO_2$, CaO and $3CaO.Al_2O_3$ are all decreasing, the $2CaO.SiO_2$ partially by combination with CaO and partially by dissolving along with $3CaO.Al_2O_3$ in the flux. As the temperature is raised still further, the amount of flux (liquid) increases and the rate of combination of CaO with $2CaO.SiO_2$ to form $3CaO.SiO_2$ increases. But it is not necessary to raise the temperature until the charge is completely melted, as normal cement clinker is obtained at temperatures much below complete melting; in other words, the necessary reactions will go to completion below the temperature required for complete melting. The rapidity with which the reactions go to completion is governed by the temperature and by the amount of flux formed at that temperature. The requisite amount of flux in turn depends upon the fineness of the raw materials, since the finer these materials are ground

the more readily the components will combine. For finely ground raw materials of the above composition, composed only of CaO, Al_2O_3, SiO_2, a temperature of about 1650° C. is required for burning. At this temperature the clinker would be about 30 per cent melted and 70 per cent solid crystal-

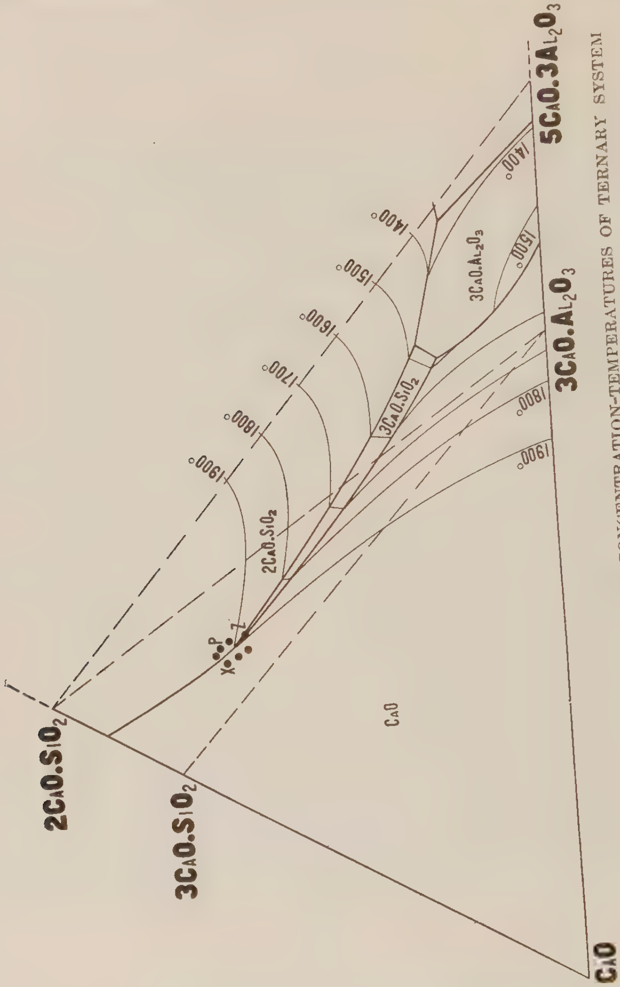

FIG. 2—PROJECTION OF A PORTION OF CONCENTRATION-TEMPERATURES OF TERNARY SYSTEM $CaO-Al_2O_3-SiO_2$ WITH ISOTHERMALS.

The dots represent the mixtures of CaO, Al_2O_3 and SiO_2 from which Portland cement of good quality can be made.

line material, a proportion of flux which would admit of the necessary reactions going to completion in a reasonable time. The charge will always completely crystallize on cooling; the percentage composition (based on actual data) of the clinker thus obtained would be approximately $3CaO.SiO_2$,

45 per cent; $2CaO.SiO_2$, 35 per cent; and $3CaO.Al_2O_3$, 20 per cent. The melting temperature of the flux necessary for the production of the clinker is materially lowered by the presence of small amounts of impurities; that the small amounts of Fe_2O_3, MgO, etc., in commercial cement actually have this effect is shown by the fact that the temperature required for burning is about 1425° C.

In the foregoing discussion, we have followed to completion the course of the reactions which take place when cement clinker composed of pure CaO, Al_2O_3, SiO_2, is burned; in other words, we have shown the formation of the compounds during the burning of mixtures of these three oxides in the proper proportions for cement clinker, and stated what compounds will be present in the final product if the burning is continued long enough and at a sufficiently high temperature.

TABLE I.—COMPOSITIONS AND BURNING TEMPERATURES OF VARIOUS PORTLAND CEMENTS.[1]

Percentage Composition of Clinker.

Portland Cements.	Actual Components.		Relative to Content of $CaO-Al_2O_3-SiO_2$.	Burning Temperature, Deg. C.	Constituents of Resulting Cements.	
Pure (P)	CaO .... 68.4 Al_2O_3 .... 8.0 SiO_2 .... 23.6	100.0	CaO ... 68.4 Al_2O_3 .. 8.0 SiO_2 ... 23.6	1650	$2CaO.SiO_2$ $3CaO.SiO_2$ $3CaO.Al_2O_3$	
White (A)	CaO .... 66.2 Al_2O_3 .... 6.4 SiO_2 .... 25.0 MgO, Fe_2O_3, Na_2O and K_2O 2.4	97.6	CaO ... 67.9 Al_2O_3 .. 6.5 SiO_2 ... 25.6	1525	$2CaO.SiO_2$ $3CaO.SiO_2$ $3CaO.Al_2O_3$	Small amount of CaO.
Gray (B)	CaO .... 63.2 Al_2O_3 .... 7.7 SiO_2 .... 22.4 MgO, Fe_2O_3, Na_2O, K_2O and SO_3 .... 6.7	93.3	CaO ... 66.7 Al_2O_3 .. 9.0 SiO_2 ... 24.3	1425	$2CaO.SiO_2$ $3CaO.SiO_2$ $3CaO.Al_2O_3$	Small amounts of $5CaO.3Al_2O_3.CaO$ and ferrites.

[1] The data given in this table are based largely on the work of this laboratory. The analyses of commercial clinkers are from publications from the Bureau of Standards and from "Portland Cement," by R. K. Meade. The temperatures of burning and the constituents given are based both on our work and that of the Bureau of Standards.

This description of the essential reactions which take place when cement made up only of CaO, Al_2O_3, SiO_2 is burned, applies equally well to commercial Portland cement. In commercial cements, however, there is always present small amounts of Fe_2O_3, MgO, alkalies, etc. These minor components, which total less than 10 per cent, have but little effect on the major constituents of the clinker. During the burning of cement clinker, however, these minor components play an important part, since their presence ensures the formation of a flux at a much lower temperature, and thereby materially promotes the combination of CaO with Al_2O_3 and SiO_2.

In order to afford a comparison of the chemical compositions, the temperature required for burning and the final products obtained for different types of Portland cement, the necessary data have been collated in Table I. The examples given in this table are based on the average for a large number

of analyses of each of three types of Portland cement, viz., pure cement, made only of CaO, Al_2O_3, SiO_2; commercial white cement; and the more common gray variety of Portland cement.

If the raw material for pure cement is perfectly burned at a temperature of 1650° C., the clinker obtained will consist of the three compounds—orthosilicate of lime, tricalcic silicate, and tricalcic aluminate. The example of a pure cement, given in Table I, has the chemical composition 68.4 per cent lime, 8.0 per cent alumina, 23.6 per cent silica. The raw material for white commercial cement, when burned at a temperature of 1525° C., will produce a clinker which consists largely of the same three compounds found in pure clinker, except for a small amount of free lime. The average chemical composition of this type of cement as given in the table is 66.2 per cent lime, 6.4 per cent alumina, 25 per cent silica, and 2.4 per cent magnesia, iron oxide and alkali. The clinker obtained on burning the raw material for commercial gray cement at 1425° C. will consist largely of the same three compounds found in the other two types of clinker, except for small amounts of free lime, the compound $5CaO.3Al_2O_3$, and iron oxide as ferrites. The composition of this clinker is 63.2 per cent lime, 7.7 per cent alumina, 22.4 per cent silica, 6.7 per cent MgO, Fe_2O_3, alkali and SO_3.

The similarity of these three types of cement clinkers is not surprising if we consider their chemical compositions. The content of lime, alumina, silica of each type is over 90 per cent, while the composition relative only to these three oxides approaches a constant, the maximum difference being 2.5 per cent in the case of alumina, since there is 6.5 per cent in white cement and 9.0 per cent in gray cement. We should expect, therefore, and it has been found experimentally to be true, that these three types of cement clinker are made up largely of the same constituents; these are, as we have shown, tricalcic silicate, dicalcium silicate, and tricalcic aluminate, all compounds of the three major components of cement.

Having shown that the components of Portland cement are CaO, Al_2O_3, SiO_2 in certain rather definite proportions and that the constituent substances are definite compounds of these oxides, let us consider the percentage of these compounds in the clinker. For example, let us take the average gray cement whose chemical composition has been given in Table I. If the clinker for this cement has been perfectly burned, it will consist of about 36 per cent $3CaO.SiO_2$, 33 per cent $2CaO.SiO_2$, 21 per cent $3CaO.Al_2O_3$ and 10 per cent of the minor constituents.

In the actual manufacture of Portland cement, however, the clinker is not always perfectly burned, that is, the raw materials are not always ground fine enough, or heated to a sufficiently high temperature so that the chemical reactions are completed. The proportions of the constituents in commercial cement will then be somewhat different from those given. With our present knowledge of the nature of the chemical reactions, however, it is possible to state which of the constituents will not be completely formed. It will be remembered that, in the discussion of these chemical reactions, it was shown that $3CaO.SiO_2$ is the last constituent to form completely and this compound is formed by combination of CaO with the compound $2CaO.SiO_2$.

It is evident, therefore, that when commercial clinker is not perfectly burned, there is less $3CaO.SiO_2$ and more $2CaO.SiO_2$ and CaO will be present as an individual constituent. In the example given there will be less than 36 per cent $3CaO.SiO_2$, more than 33 per cent $2CaO.SiO_2$ and there will be a certain percentage of free CaO. The exact percentages will of course depend upon how near to completion the reaction, $CaO+2CaO.SiO_2=3CaO.SiO_2$, has been carried.

That the manufacture of good Portland cement necessitates that this reaction be carried practically to completion is evident if we consider certain facts in regard to the influence of lime on the physical properties of Portland cement. Practical experience has shown that cements containing much free lime are unsound and that concrete made from them will in time disintegrate. This is due to the expansion of free or uncombined lime when it reacts with water to form calcium hydrate. If, however, the lime in cements is all combined, they are sound and of good strength. The importance of the reaction $CaO+2CaO.SiO_2=3CaO.SiO_2$ is, therefore, apparent, since this reaction must go practically to completion in order that a sound cement may be produced. It has long been recognized that anything which will promote the combination of lime during burning will promote soundness in cement and that the greater the percentage of combined lime the greater the strength of the cement.

The average lime content of cement today is about 62.5 per cent, which is largely combined as $3CaO.SiO_2$, $2CaO.SiO_2$ and $3CaO.Al_2O_3$. If the percentage of CaO were increased, it would tend to combine with the $2CaO.SiO_2$ to form more $3CaO.SiO_2$ and would so combine if the time of burning were long enough and the temperature sufficiently high. Since practical experience has shown that increased percentage of lime increases both the percentage of $3CaO.SiO_2$ and the strength of cements, it may be inferred that the strength of cements is largely due to the compound $3CaO.SiO_2$. If this is true, it is desirable that Portland cement should contain as high a percentage of this compound as is possible. An average Portland cement contains about 30 to 35 per cent of this constituent. That Portland cement contains such a small amount of $3CaO.SiO_2$ is due partly to the fact that this constituent is formed with great difficulty and also to the fact that about 40 per cent is the maximum yield which could be obtained from raw materials having the same CaO, Al_2O_3, SiO_2 composition as are now used.

Before taking up, however, a discussion of the probable value of $3CaO.SiO_2$ as a cementing material and the possibility of increasing its percentage in Portland cement, let us consider what is known as to the cementing value of the constituents of Portland cement, taking up first the changes which take place when Portland cement is mixed with water and hardens.

When Portland cement is finely pulverized and mixed with water, a hard mass is formed by chemical action between the water and the constituents of the cement. While there is still much to be learned as to the chemistry of the hardening of Portland cement, sufficient data on the hydration of the individual major constituents have been obtained to enable us to

account for the gradual hardening and increase in strength and to indicate the relative value of these constituents as cementing materials.

Let us now consider the hydration of the three major constituents $3CaO.Al_2O_3$, $3CaO.SiO_2$, $2CaO.SiO_2$ in the order named. When pure $3CaO.Al_2O_3$ is mixed with water, an amorphous hydrated material is first formed. This material sets and hardens very rapidly. The compound $3CaO.SiO_2$ when mixed with water, also sets and hardens rather rapidly. In the case of this compound, as in the case of $3CaO.Al_2O_3$, the setting and hardening is due to the formation of an amorphous hydrated material on the individual grains which are thus cemented together. The extent of the hydration or the percentage of amorphous material which each grain will yield depends upon the percentage of water used and the time. With a given percentage of water the amount of amorphous material formed from the compound $3CaO.Al_2O_3$ in a given time is much greater than for the compound $3CaO.SiO_2$, that is, the compound $3CaO.Al_2O_3$ reacts with water much more rapidly than the $3CaO.SiO_2$. The compound $2CaO.SiO_2$ reacts very slowly with water and it is only after a long period of time that sufficient amorphous hydrated material is formed to cement together the grains of this compound and so form a hard mass.

The amorphous hydrated material formed by the action of water on the constituents of cement, does in time, no doubt, crystallize to some extent. From the data available it would appear that the crystals formed are calcium hydrate and some crystalline hydrate derived from $3CaO.Al_2O_3$. Apparently no crystalline hydrate of the calcium silicates is formed.

From this brief description of the action of water on the constituents of Portland cement, it will be seen that the setting and hardening of Portland cement involves the formation of an amorphous hydrated material which subsequently partially crystallizes; that the initial set is probably due to the hydration of $3CaO.Al_2O_3$; that the hardness and cohesive strength at first are due to the cementing action of the amorphous material produced by the hydration of this aluminate and of the $3CaO.SiO_2$; and that the gradual increase in strength is due to further hydration of these two compounds together with the hydration of the $2CaO.SiO_2$.

Of the three compounds which thus take part in the setting and hardening of Portland cement, the $3CaO.SiO_2$ appears the best cementing constituent; that is, this compound is the only one of the three which when mixed with water will set and harden within a reasonable time to form a mass which in hardness and strength is comparable to Portland cement. The compound $2CaO.SiO_2$ requires too long a time to set and harden, in order to be in itself a valuable cementing material. The compound $3CaO.Al_2O_3$, while it sets and hardens rapidly, is rather soluble in water and is not particularly durable or strong.

From this, again, it follows that the compound tricalcium silicate is *the* essential constituent of Portland cement, consequently the higher its percentage the better the cement. Granting for the time being that this is true, let us consider the nature of an investigation which might lead to the

production of a cement containing a much higher percentage of $3CaO.SiO_2$ than is contained in Portland cement as made today.

In the discussion of the constitution of Portland cement we have shown that an average Portland cement contains about 30 to 35 per cent tricalcium silicate, a proportion which closely approaches the maximum possible yield if the components are in the proportions of an average Portland cement. It has also been shown that an increase in the lime content of an average cement will increase the percentage of $3CaO.SiO_2$ if the conditions of burning are such that the reaction $CaO+2CaO.SiO_2=3CaO.SiO_2$ goes to completion. This, however, necessitates finer grinding of the raw materials, as well as burning for a longer time and at an increased temperature, factors which materially affect the cost of production. Now the data discussed above were obtained by applying the results obtained by an investigation of the equilibrium relations found to exist in the system CaO-Al_2O_3-SiO_2, to the actual manufacture of Portland cement; but this by no means implies that in presence of other components the conditions required for the production of an adequate amount of flux should not be more favorable and economical. In other words, the study of other systems may establish the economic possibility of producing a cement containing a high percentage of tricalcium silicate. For example, if some substance were substituted for the component Al_2O_3, in the system CaO-Al_2O_3-SiO_2, the study of the equilibrium relations found to exist in this new system would enable one to determine whether or not it would be economically possible to produce a cement containing a high percentage of $3CaO.SiO_2$ from raw materials of which the components are CaO, SiO_2, and this third substance. Thus if Fe_2O_3 were substituted for Al_2O_3, we could from the study of the system CaO-Fe_2O_3-SiO_2 ascertain the fineness of the raw material, and the time and temperature of burning, necessary to secure a clinker containing the highest percentage of $3CaO.SiO_2$ which could be economically produced from raw materials of which the major components are CaO, Fe_2O_3, SiO_2. This would require that one determine the nature of all compounds formed in mixtures of these three oxides which, when burned, contain $3CaO.SiO_2$, and that we establish the identity, melting temperature, and rate of formation, of $3CaO.SiO_2$ in such mixtures. Instead of substituting a single substance, it would undoubtedly be more desirable to substitute a number of different substances, since the presence of several produces a lower melting flux and thus makes possible the formation of $3CaO.SiO_2$ at a lower temperature. By proceeding in this way to determine systematically the various mixtures of substances, which, when burned, give high percentages of $3CaO.SiO_2$, it would not seem at all improbable that we may discover some mixture which could be economically manufactured and which would result in the production of a cement far superior to the Portland cement now made.

In this discussion we have assumed that tricalcium silicate is the essential constituent of Portland cement. If subsequent investigation should show that other constituents possessed superior cementing qualities, the nature of an investigation to determine the mixture which would economically produce the highest percentage of such a constituent, would be of the same

general nature as that described for tricalcium silicate. Indeed, the determination of the constituents (compounds) which possess the best cementing qualities for various purposes may be determined in just this way, since the systematic study of the components found in cements enables one to isolate the separate compounds formed and to determine their cementing qualities pure and in mixtures.

In conclusion, let us recapitulate the main points contained in this paper. The value of Portland cement depends upon the fact that when finely powdered and mixed with water it forms a hard mass; and the strength and permanence of this mass depend upon the constituents of the cement. The major constituents are tricalcium silicate, dicalcium silicate and tricalcium aluminate. Of these constituents, the compound tricalcium silicate is the one which hardens and develops the greatest strength within a reasonable time. This most important constituent, which is the one formed with the greatest difficulty, makes up only about 30 to 35 per cent of an average normal Portland cement. It may be said, therefore, that the essential process for the manufacture of Portland cement is the formation of this compound, and that any improvement in this process yielding an increased percentage of tricalcium silicate will increase the cementing value of Portland cement. In order to determine the most economical process for producing tricalcium silicate in the highest percentages, it will be necessary to study the rate of formation of this compound in series of mixtures of various substances; this, in turn, necessitates the determination of the equilibrium relations of tricalcium silicate at high temperatures in such mixtures. Such a procedure will lead sooner to the discovery of the optimum composition in various cases and for various purposes than the empirical cut-and-try methods which hitherto have been the only method tried.

ANNUAL REPORT OF BOARD OF DIRECTION.

At the close of the Convention in February, 1915, a new president was elected, and several changes made in the Board of Direction. The new officers had some difficulty in getting started in their annual work. The condition of the papers and other affairs at headquarters had to be straightened out, and this took several months. In the spring and early summer several numbers of the JOURNAL, from March to August, 1915, inclusive, and also for March, 1914, were published. In August, Mr. C. L. Fish resigned the office of secretary, and since September 1, Mr. John M. Goodell has served, without compensation, as acting secretary, doing most valuable and important work, and the Institute has flourished under his management.

Due to a number of circumstances entirely beyond our control, various sources of income which we anticipated would be received were not forthcoming, very materially reducing our resources. It has, therefore, been necessary to discontinue the publication of the monthly JOURNAL, as this exceeded our resources. We shall, in the future, publish from time to time at irregular intervals reports of committees, and such other matter as we can, and circulate them among our members. A number of meetings of the Executive Committee and of the Board of Direction have been held during the year. The Treasurer's report is submitted for your consideration.

It has been decided to discontinue the office in Philadelphia and all paid employees. The President has consented to furnish the Institute, free of charge, the use of his offices in Boston to conduct the affairs of the Institute. The activities of the Institute, therefore, will not be handicapped by the change in office location, and regular meetings of the officers will be held and the work of committees will be continued.

LEONARD C. WASON,
President.

JOHN M. GOODELL,
Acting Secretary.

ANNUAL REPORT OF TREASURER.

Under Article IV, Section 1, of the By-Laws of the Institute, its fiscal year begins on July 1, and under Article II, Section 6, the accounts of the Secretary and Treasurer are audited annually. In accordance with these requirements, the accounts were audited, as of July 1, 1915, by Lybrand, Ross Bros. & Montgomery, Certified Public Accountants, of Philadelphia.

They reported a bank balance of	$1,796.75	
and a cash balance of	100.00	
		$1,896.75
In the period between July 1, 1915, and February 1, 1916, the receipts have been		7,805.39
		$9,702.14

NOTE.—Of this amount $800 has been paid in for specified purposes and is not available for current Institute expenses.

The expenditures during the period from July 1, 1915 to February 1, 1916, were	$7,494.26
Balance, February 1, 1916	$2,307.88

NOTE.—Of this amount, $100 was cash on hand and $800 more was available only for specific purposes named by donors.

PHILADELPHIA, PA., 20th August, 1915.

American Concrete Institute,
Bellevue Court Building,
Philadelphia.

DEAR SIRS:

We report that we have audited the accounts of the Institute for the period from 13th February, 1915, to 30th June, 1915, and with the exception of a few clerical errors which were adjusted during the course of our audit found them to be correct.

Of the uncollected subscriptions amounting to $3,375.00, as shown by the annexed statement, $1,450.00 are pledged with signatures of the pledgers on the subscription list and $1,925.00 without signatures on the subscription list.

The subscriptions not definitely pledged amounting to $1,350.00 are not entered in the books of the Institute.

We submit herewith the following statements:

Balance Sheet, 30th June, 1915.

Statement of Receipts and Disbursements for Year ended 30th June, 1915.

Uncollected Subscriptions, 30th June, 1915.

Very truly yours,

(Signed) LYRAND, ROSS BROS. & MONTGOMERY.

BALANCE SHEET, 30TH JUNE, 1915.

ASSETS.

Cash:		
In bank	$1,796.75	
On hand	100.00	
		$1,896.75
Accounts receivable:		
Dues, etc. (subject to adjustment)	$7,657.65	
Subscriptions	3,375.00	
C. L. Fish	135.00	
		11,167.65
Inventories:		
Proceedings	$2,781.49	
Journals	1,026.00	
Membership pins and certificates	94.05	
Miscellaneous reports and standards	101.25	
Furniture and fixtures	244.50	
Printing and stationery	77.50	
		4,324.79
		$17,389.19

LIABILITIES.

Accounts Payable:		
John C. Winston Co	$816.97	
R. L. Humphrey	1,147.79	
Lybrand, Ross Bros. & Montgomery	300.00	
E. E. Krauss	17.57	
National Fire Protective Association	25.00	
William Dawson Sons	5.00	
E. A. Wright Bank Note Co	1.10	
Colonial Springs Co	1.00	
Liberty Typewriter Co	2.50	
		$2,316.93
Advances from members and non-members for bindings, proceedings, dues, etc		136.79
Surplus, subject to adjustment of accounts receivable		14,935.47
		$17,389.19

STATEMENT OF RECEIPTS AND DISBURSEMENTS
For the Year ended 30th June, 1915.

RECEIPTS.

Balance, July 1, 1914			$41.67
Dues		$1,360.65	
Publications:			
Sales, Proceedings	$330.71		
" Standards	887.84		
" Journals	41.00		
" Membership certificates	29.50		
		1,289.05	
Subscriptions		14,125.00	
Advance by R. L. Humphrey		175.00	
Miscellaneous		7.27	
			16,956.97
Total receipts			16,998.64

DISBURSEMENTS.

Salaries		$2,911.90	
Office rent		820.85	
Printing, etc.:			
Proceedings	$3,688.09		
Journals	2,870.08		
Standards, reports, circulars, etc	860.57		
		7,418.74	
Chicago Convention expenses		1,617.85	
R. L. Humphrey's expenses incurred obtaining subscriptions		1,000.00	
Membership Buttons and Certificates		172.75	
Storage		100.00	
Research Fund and Contingencies		299.54	
Auditing		70.00	
Office and miscellaneous expenses		689.76	
Total Disbursements			$15,101.89
Balance June 30, 1915:			
In bank		$1,796.75	
On hand		100.00	
			$1,896.75
			$16,998.64

MINUTES OF MEETINGS OF THE BOARD OF DIRECTION.

Meeting of the Board of Direction, Held at Auditorium Hotel, Chicago, Ill., February 10, 1915.

Present: Wason, Anderson, Boyer, Hatt, Humphrey, Lindau, Turner and Fish.

Upon motion, it was voted that Mr. Charles L. Fish be elected Secretary of the Institute, to take office at the close of the Convention.

Upon motion, it was voted that Edward D. Boyer and Henry C. Turner become members of the Executive Committee to serve with the President, Treasurer, and Secretary *ex officio*.

Upon motion, it was voted that the Finance Committee be composed of Leonard C. Wason, Henry C. Turner and Robert W. Lesley.

Upon motion, it was voted that the thirteen regular Technical Committees of the Institute be continued with their present chairmen, with the exception that Mr. Cloyd N. Chapman succeed Mr. Leonard C. Wason as Chairman of the Committee on Concrete Surfaces. The President to consult with the various chairmen as to the personnel of their committees.

Upon motion, it was voted that the Secretary be bonded in amount of $2,500, the bond premium being paid by the Institute.

Upon motion, it was voted that the Executive Committee be allowed to determine the methods of disbursing funds.

Upon motion, it was voted that a Committee of Three be appointed to make a report, outlining a program of research work.

Upon motion, it was voted that the books of the Institute be closed as of date February 13, 1915, and audited by Lybrand, Ross Bros. & Montgomery.

Upon motion, the meeting adjourned.

Meeting of the Executive Committee, Held at 11 Broadway, New York, N. Y., April 23, 1915.

Present: Wason, Boyer, Turner and Fish.

Depository of Funds.—Upon motion, it was voted that the Depository of the Institute be the Girard Trust Company of Philadelphia, and that the signatures required for the withdrawal of such funds shall be those of the President, Treasurer and Secretary.

Subscription to Journal.—Upon motion, it was voted that the subscription price to the Journal of the Institute be in amount equal to the membership dues.

Time of Holding Convention.—It was unanimously agreed that the month of February be established as the time of holding the Annual Convention of the Institute.

Gift of Past Proceedings to Subscribers.—It was unanimously agreed that the Institute present to the various subscribers to the fund, cloth-bound volumes of past Proceedings beginning with Volume IV, and that same be accompanied by an appropriate letter from the President.

Nominating Committee.—It was unanimously agreed that the May number of the Journal contain a letter ballot for committee of five members on Nomination of Officers and that a return envelope be included in same.

Adjournment.

MEETING OF THE BOARD OF DIRECTION, HELD AT HOTEL TRAYMORE, ATLANTIC CITY, N. J., June 23, 1915.

Present: Wason, Anderson, Ashton, Boyer, Hatt, Humphrey, Lesley and Fish.

Leonard C. Wason Medal.—Upon motion, it was voted that Mr. Richard L. Humphrey act with the chair in appointing the Committee of Award of the Leonard C. Wason Medal.

Upon motion of Mr. Lesley and seconded by Mr. Boyer, it was voted that the word "nickel" in paragraph 2 of the Rules Governing the Award of said Medal be changed to read "Silvered Bronze."

Preliminary sketches of the Leonard C. Wason Medal were submitted by the President for consideration. It was agreed that the members present meet at 11 o'clock, July 24, for the purpose of giving these sketches further consideration.

Committee on Publicity.—Upon motion, it was voted that Mr. Lesley be appointed Chairman of the Committee on Publicity to outline a policy to be adopted by said Committee.

Giving Papers to Press.—Upon motion, it was voted that hereafter no authority be granted for the full publication of any papers, reports or communications until after same have been published by the Institute, but that the Committee on Publications and Papers be authorized to abstract any such papers, reports or communications for publication in any papers other than the Journal of the Institute.

Committee on Research.—It was the sense of the meeting that the Chair appoint a Committee on Research consisting of the President together with the Chairman of the Committees on Reinforced Concrete and Building Laws and Concrete Aggregates.

Building Block and Cement Products.—It was the sense of the Board that Mr. R. F. Havlik, of Moosehart, Ill., be appointed Chairman of this Committee, providing he accepts same.

Insurance.—It was unanimously agreed that the present Chairman of this Committee, Mr. J. P. H. Perry, be reappointed.

Nomenclature.—It was unanimously agreed that Mr. Frank C. Wight, the present Chairman of this Committee, be reappointed.

Reinforced Concrete and Building Laws.—It was unanimously agreed that Prof. Arthur W. French of the Worcester Polytechnic Institute or Mr. Charles

Killam of Boston be appointed Chairman of this Committee. Same to be left to the discretion of the President.

Reinforced Concrete Highway Bridges and Culverts.—It was unanimously agreed that Mr. C. B. McCullough, Designing Engineer, Iowa State Highway Committee, Ames, Iowa, be appointed Chairman of this Committee, contingent upon his acceptance of same.

Concrete Roads and Pavements.—It was unanimously agreed that the present Chairman, Mr. A. N. Johnson, be continued, together with the present Secretary, Mr. W. M. Kinney.

Concrete Aggregates.—It was unanimously agreed that Mr. Sanford E. Thompson be reappointed Chairman of this Committee.

Reinforced Concrete Chimneys.—It was unanimously agreed that Mr. Skinner be appointed Chairman of this Committee, contingent upon his acceptance of same.

Treatment of Concrete Surfaces.—It was unanimously agreed that Mr. Cloyd M. Chapman be continued as Chairman of this Committee.

Fireproofing.—It was unanimously agreed that Mr. John S. Sewell be appointed Chairman of this Committee.

Cement Roofing Tile.—It was unanimously agreed that Mr. Allen, Superintendent of Buildings, Cleveland, Ohio, be appointed Chairman of this Committee, contingent upon his acceptance of same.

Sidewalks and Floors.—It was unanimously agreed that Mr. P. M. Bruner, of the P. M. Bruner Granitoid Co., St. Louis Mo., be appointed Chairman of this Committee.

Use of Concrete in Revetment Work.—Mr. W. P. Anderson volunteered to furnish the President with information relative to the party who had been in charge of this work in Cincinnati.

Steel for Reinforcing.—It was unanimously agreed that this Committee be merged with the Committee on Reinforced Concrete and Building Laws.

Form of Specifications.—It was unanimously agreed that the Committee on Publications and Papers, determine a form of Specifications as the Institute might issue.

Proceedings to be put into Book Stores.—It was unanimously agreed that the Proceedings of the Institute be placed with book stores in the large cities of the country for sale on such terms as might be favorably made.

Financial Statement to Board.—It was unanimously agreed that a monthly statement of the finances of the Institute be sent to all members of the Board of Direction.

Method of Committee Report. Adoption of A. S. T. M. Method for Voting on Specifications.—It was unanimously agreed that this method be adopted.

Bills for Current Year.—It was unanimously agreed that the bills for the current fiscal year 1915–1916 be sent to members August 1, 1915.

Correspondence with National Association of Builders' Exchange.—It was unanimously agreed that Messrs. Hatt, Anderson and Ashton attend the Convention to be held on or about August 1, 1915.

Adjournment.

MEETING OF THE EXECUTIVE COMMITTEE, HELD AT THE OFFICE OF THE INSTITUTE, July 21, 1915.

Present: L. C. Wason, E. D. Boyer, R. W. Lesley, H. C. Turner and C. L. Fish.

Account of Richard L. Humphrey.—Upon motion made by Mr. Boyer and seconded by Mr. Turner, the account of Mr. R. L. Humphrey under date of May 15, 1915, together with statement and vouchers accompanying same, showing a balance in his favor of $2,180.54, was approved. The President stated that payment of $1,000 on this account was made to Mr. Humphrey by voucher No. 36, June 12, 1915.

Valuation of Inventory.—Valuation of the inventory of books, publications, furniture and other property of the Institute, showing a value of $4,319.79, was presented. Upon motion, made by Mr. Lesley and seconded by Mr. Turner, it was directed that the Treasurer enter this inventory on the books of the Institute at the valuation given above.

Adjournment.

MEETING OF THE BOARD OF DIRECTION, HELD AT THE HOUSE OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS, NEW YORK CITY, AUGUST 4, 1915.

Present: L. C. Wason, H. C. Turner, R. W. Lesley, R. L. Humphrey and C. L. Fish.

Trunk Line Suit.—Mr. R. L. Humphrey stated that the attorney for the Institute in the Trunk Line Suit reported that his bill against the Institute, in amount $50, had been paid by the Cement Products Exhibition Company.

Resignation of C. L. Fish.—Upon motion, made by Mr. R. W. Lesley and seconded by Mr. H. C. Turner, the resignation of C. L. Fish, Secretary of the Institute, was accepted to take effect September 1, 1915.

Acting Secretary.—The President reported a conversation with Mr. John M. Goodell and on motion, made by Mr. H. C. Turner and seconded by Mr. R. W. Lesley, the President was empowered to negotiate with him to become "Acting Secretary."

Report of Finance Committee.—Mr. H. C. Turner presented a report of the Finance Committee and same was duly approved.

Report of Auditors.—Mr. R. W. Lesley, Treasurer, presented the report of Messrs. Lybrand, Ross Bros. & Montgomery, as of May 25, 1915, and stated that their report as of July 1, 1915, would be presented within a few days.

Meeting adjourned.

JOINT MEETING OF THE EXECUTIVE AND FINANCE COMMITTEES AT 11 BROADWAY, NEW YORK, NOVEMBER 4, 1915.

Present: Wason, Turner, Lesley and Goodell.

An analysis of the financial condition and possible early revenues of the Institute was made and the following statement adopted as expressing the views of those present:

Expenses.		Possible Revenue.	
Bills payable, Nov. 1	$2,475	Bank balance, Nov. 1	$1,881
August Journal	700	Subscriptions	1,000
Other Journals, 4 at $400	1,600	Dues, 100 at $10	1,000
Journals for 1914	1,000	Sustaining members, 50 at $30	1,500
Membership campaign	200	Office sales	100
Total	$7,575	Total	$5,481
		Possible deficit	2,094

It was voted to pay the following bills incurred up to November 1: John C. Winston Co., $752.88; Andrew L. Logan & Co., $2.25; W. S. Best Printing Company, $5.00, Bell Telephone Company $12.56; E. A. Wright Bank Note Co., 95 cents; Secretary's salary, $200; petty cash, $135.55. Payment of all other accounts payable was deferred for further consideration.

The Acting Secretary was directed to mail the August number of the Journal, but to send no more copy to the printer without directions to do so from the Executive Committee or the Board of Direction. The preparation of the manuscript of the remaining 1914 and 1915 Journals was to be pushed as rapidly as possible in order to have it ready for immediate use when needed.

The Acting Secretary was instructed to confer with the leading members of the Association of American Portland Cement Manufacturers to ascertain if they would favor the granting of financial aid to the Institute by the Association for a period of one year on the ground of the important publicity work done by the Institute for the cement industry, this aid to be given for definite objects and paid on vouchers presented by the Institute.

In order that the condition of the Institute might be known to each of its officers and its needs discussed in complete detail, it was voted to call a meeting of the Board of Direction in New York on November 18th for the discussion of very important business.

Meeting adjourned.

Meeting of Board of Direction, Held at 11 Broadway, New York, November 18, 1915.

Present: Wason, Lesley, Anderson and Goodell; Mr. Humphrey was present later for part of the session.

President Wason outlined the Institute's financial history for the last two years and stated that the Institute was now in debt as shown by the following figures:

Current accounts payable:	
Printer	$673
Richard L. Humphrey	680
Lybrand, Ross Bros. & Montgomery	375
Walter B. Snow	521
Petty cash	74
Total unpaid bills	$2,323

December rent, payable in advance		$42
November salaries		200
December salaries		200
Miscellaneous		50
Minimum accounts payable to end of 1916		$2,815
September, 1915, Journal		400
October, 1915, Journal		400
Minimum expense with continuation of Journals		$3,615
Bank balance, November 15th	$1,247	
Possible income from dues and subscriptions	368	
		1,615
Sum to be raised		$2,000

President Wason submitted a statement of the work done by Walter B. Snow in collecting arrears of dues and securing new members.

The Acting Secretary stated that he hoped to be able to report in a month on the practicability of obtaining financial cooperation in the Institute's work during 1916 from outside sources. It was decided to defer publication of any number of the Journal until funds for the purpose were available, to stop the campaign for new members, and to continue the work by Walter B. Snow under the President's direction for collecting dues in arrears.

Messrs, Wason, Turner and Anderson authorized the Acting Secretary to state to possible financial backers of the Institute that an early meeting of concrete contractors would be called, at which the donation of funds for issuing the September, October and November, 1915, Journals would be discussed. It was decided to hold the completion of the 1913 and 1914 Proceedings in abeyance until the condition of the Institute for 1916 was settled.

Mr. Humphrey stated that his account against the Institute could hold over for the present.

It was decided to send a letter to each director pointing out that a deficit existed, amounting to $1,200 by the end of December, 1915, and in the opinion of those present at this meeting it was desirable to have it met by personal contributions of the directors, an early statement of the contribution each director would make being desired.

The Acting Secretary was instructed to dispose of the stock of pamphlets which had been superseded by revised editions but were carried as assets on the inventory.

It was decided to hold a meeting of the Board in New York during the week beginning December 12, 1915, the exact date to be fixed by the Acting Secretary.

MEETING OF BOARD OF DIRECTION, HELD AT 11 BROADWAY, NEW YORK, N. Y., JANUARY 6, 1916.

Present: Wason, Turner, Lesley, Humphrey, Ham and Goodell.

The Acting Secretary explained the steps taken to secure from the American Association of Portland Cement Manufacturers financial support

for the Institute during 1916, and stated that the Advisory Committee of the Association had declined to grant such aid.

The financial condition of the Institute was explained and discussed.

It was voted to discontinue the office of the Institute and all salaried employes after February, 1916.

It was voted to accept the offer of President Wason to store without charge 25 copies of each of the publications of the Institute.

It was voted to suspend the publication of the Journal.

It was voted to accept the offer of President Wason to send various publications of the Institute to members to be designated by him, packing and forwarding charges to be paid by him.

It was voted to hold a convention at Chicago on February 14 to 19 inclusive, and a telegram was sent to B. F. Affleck asking for a contribution toward the expense of the Convention from the Cement Products Exhibition Company.

A preliminary program of the Convention was adopted and the Acting Secretary was instructed to issue at once a circular regarding the Convention.

MEETING OF THE BOARD OF DIRECTION, HELD AT CHICAGO, ILL., FEBRUARY 15, 1916.

Present: Wason, Hatt, Turner, Lesley, Anderson, Boyer, Lindau and Goodell.

Messrs. Turner, Humphrey, Boyer and Lindau were authorized to endeavor to collect certain unpaid subscriptions to the funds of the Institute.

The Secretary was instructed to present the resignation of the Institute from the National Fire Protective Association and the American Society for Testing Materials.

The Treasurer was authorized to pay at once the bill of the John C. Winston Co. for $201; that of John M. Goodell for money advanced for petty cash for $47.07; that of the convention stenographer for $250; that of Lybrand, Ross Bros. & Montgomery for $375; that of the Aberthaw Construction Company for $15; that of Walter B. Snow for $365.10; that of Henry C. Turner for $7.64; and that of the Turner Construction Company for $112.08; and to pay as funds became available that of John M. Goodell for $387.38; of Richard L. Humphrey for $724.56; and those incurred in closing up the Philadelphia office and the expenses of the convention.

It was voted to pay interest on any outstanding accounts.

The President was authorized to dispose of the furniture in the office at Philadelphia at the best price obtainable.

The President was authorized to accept an offer from F. H. Price of the Free Public Library of Philadelphia to take from their present places of storage all publications of the Institute, without charge to the Institute, and to dispose of them by exchange with other libraries and owners of books who exchange volumes.

It was voted to move the office of the Institute from Philadelphia to Boston.

It was voted to remit all unpaid dues of members who had paid their dues for 1915–1916 but had not paid dues for years when the publications of the Institute were incomplete.

The annual report for presentation to the annual business meeting on February 16 was adopted.

It was voted to give the manuscript of the papers and reports of the conventions of 1913 and 1914 to Richard L. Humphrey.

ATTENDANCE, TWELFTH ANNUAL CONVENTION.

D. A. Abrams, Chicago, Ill.
Wm. W. Achison, Syan, N. Y.
B. F. Affleck, Chicago, Ill.
J. W. Allan, Minneapolis, Minn.
J. H. Ames, Ames, Ia.
A. J. A. Anderson, Champaign, Ill.
W. P. Anderson, Cincinnati, O.
W. S. Anderson, Chicago, Ill.
R. C. Angevine, Cold Water, Mich.
Elhan C. Arnold, Elkhart, Ind.
Ernest Ashton, Chicago, Ill.
P. H. Atwood, Armstrong, Ia.
H. E. Babcock, Chicago, Ill.
E. B. Babel, Chicago, Ill.
W. H. Baer, Chicago, Ill.
Chas. Baker, New York, N. Y.
Douglas Banfield, Chicago, Ill.
W. B. Banning, Union, Neb.
R. C. Barler, Chicago, Ill.
J. F. Base, Maywood, Ill.
Stanley E. Bates, Chicago, Ill.
N. H. Battjes, Grand Rapids, Mich.
D. D. Battjes, Grand Rapids, Mich.
Alfred Beck, Chicago, Ill.
J. B. Beck, Philadelphia, Pa.
A. B. Becker, Chicago, Ill.
Rodney Bell, Paris, Ill.
John Benson, Chicago, Ill.
M. A. Berns, Chicago, Ill.
John J. Berscheir, Chicago, Ill.
Harry T. Bignal, Chicago, Ill.
Frank J. Billeter, Indianapolis, Ind.
S. J. Binsinger, Chicago, Ill.
L. A. Bissonnette, Chicago, Ill.
A. E. Bjosk, Chicago, Ill.
Gurdon G. Black, St. Louis, Mo.
J. H. Black, Mason City, Ia.
H. M. Blackburn, Chicago, Ill.
B. Blair, Wood Stock, Ont.
Geo. I. Blowers, Chicago, Ill.
Jos. J. Bogdes, Chicago, Ill.
Hugh Borland, Chicago, Ill.
Edw. D. Boyer, New York, N. Y.
E. W. Boynton, Muscatine, Ia.
John Bowditch, Chicago, Ill.
Geo. H. Boynton, Muscatine, Ia.
Walter C. Boynton, Detroit, Mich.
Addison Brannin, Aberdeen, Miss.
Casper Braun, Berlin, Ont.
S. E. Bradt, Dekalb, Ill.
B. W. Brayton, Waterloo, Ia.
Benj. Brooks, Kansas City, Mo.
Chas. C. Brown, Indianapolis, Ind.
Harold P. Brown, New York, N. Y.
W. Jess Brown, Chicago, Ill.
H. B. Bushness, Aurora, Ill.
G. F. Burch, Moline, Ill.
M. W. Cameron, Indianapolis, Ind.
J. H. Campbell, Chicago, Ill.
O. B. Canaday, Gary, Ind.
P. H. Carlin, New York, N. Y.
F. O. Carlson, Monroe Center, Ill.
W. P. Carmichael, St. Louis, Mo.
J. N. Cash, Monticello, Ind.
E. R. Chamberlain, Dallas, Tex.
F. Ross Chamberlain, Minneapolis, Minn.
Henry Chase, Eau Claire, Wis.
Geo. N. Childs, Chicago, Ill.
W. T. Chollar, New York, N. Y.
J. H. Chubb, Minneapolis, Minn.
Jas. E. Churchill, Chicago, Ill.
Louis R. Cobb, New York, N. Y.
L. Drew Coddard, La Porte, Ind.
Louis S. Cole, Chicago, Ill.
E. W. Collen, Madison, Wis.
W. A. Collings, Chicago, Ill.
T. L. Condron, Chicago, Ill.
Chas. F. Conn, Philadelphia, Pa.
L. J. Conter, La Grange, Ind.
Frank W. Cook, Minneapolis, Minn.
J. E. Conzelman, St. Louis, Mo.
O. A. Cost, Chicago, Ill.
W. P. Cottingham, Gary, Ind.

Horace A. Crain, Kentland, Ind.
Chas. O. Gray, Chicago, Ill.
A. R. Criffes, Cincinnati, O.
A. C. Cronkrite, Chicago, Ill.
A. J. R. Curtis, Oak Park, Ill.
E. P. Cutcher, Chicago, Ill.
J. H. Dalley, Chattanooga, Tenn.
T. M. Davidson, Chicago, Ill.
H. E. Davis, Toronto, Canada.
J. K. Davis, Knoxville, Tenn.
W. F. M. David, Miles, Wis.
Stanley Deam, Chicago, Ill.
W. W. DeBerard, Chicago, Ill.
E. H. Defebaugh, Chicago, Ill.
Major T. M. De Frees, Berkley Springs, W. Va.
F. W. De Wolf, Urbana, Ill.
Geo. P. Dickman, Mason City, Ia.
J. C. Donaldson, Chicago, Ill.
E. J. Dowdall, Chicago, Ill.
N. W. Duncam, Chicago, Ill.
Ellis R. Dutton, Minneapolis, Minn.
A. V. Eberhart, St. Paul, Minn.
A. J. Eckly, Pittsburgh, Pa.
Wm. G. Edene, Chicago, Ill.
A. B. Edwards, Chicago, Ill.
Jos. F. Eilbacher, Elizabeth, N. J.
A. W. Eisenmeyer, Jr., Granite City, Ill.
Wm. Elsberg, Minneapolis, Minn.
W. B. Elcock, Denver, Col.
Jas. A. Evarts, Minneapolis, Minn.
Davis Ewing, Bloomington, Ill.
Frederic H. Fay, Boston, Mass.
Lewis R. Ferguson, Philadelphia, Pa
E. E. Fillion, Indianapolis, Ind.
H. L. Fixmer, Chicago, Ill.
Walter R. Forbush, Newton, Mass.
J. Kyle Foster, Chicago, Ill.
A. L. Frank, Kansas City, Mo.
J. E. Freeman, Chicago, Ill.
Fred Freidline, Kentland, Ind.
Amos A. Fries, Yellowstone Park, Wyo.
Pierce P. Fuber, Chicago, Ill.
W. A. Fuchs, Allentown, Pa.
Frank M. Gaiger, La Grange, Ill.
J. J. Gaillard, Macon, Ga.
Lion Gardiner, Chicago, Ill.
W. H. Gardner, Chicago, Ill.
W. Gearhart, Manhattan, Kan.
L. S. Gelser, Chicago, Ill.
Wm. George, Aurora, Ill.
H. A. Gerdes, Minneapolis, Minn.
C. E. Geuf, Freeport, Ill.
C. D. Gilbert, Detroit, Mich.
F. B. Godley, New York, N. Y.
A. C. Godward, Minneapolis, Minn.
H. F. Gonnerman, Urbana, Ill.
John M. Goodell, Montclair, N. J.
L. F. Gottschalk, Columbus, Neb.
J. A. Granniger, Chicago, Ill.
P. J. Greaves, Chicago, Ill.
J. H. Grehan, Chicago, Ill.
C. J. Griesenaver, Chicago, Ill.
S. H. Gowdy, San Antonio, Tex.
F. M. Haas, Minott, Ill.
Arthur Hagener, Chicago, Ill.
Robt. F. Hall, Chicago, Ill.
H. D. Hallett, Aurora, Ill.
Elizah Hamilton, Chicago, Ill.
James H. Hamilton, Arkansas City, Kan.
R. R. Hamilton, Chicago, Ill.
David E. Hannan, Chicago, Ill.
C. L. Hanson, Batavia, Ill.
R. E. Hanson, Attaca, Ind.
E. S. Hanson, Chicago, Ill.
Oscar Harder, Chicago, Ill.
W. H. Harding, Philadelphia, Pa.
Richard Hardy, Chattanooga, Tenn.
W. E. Harrison, Chicago, Ill.
Geo. C. Haun, Chicago, Ill.
W. K. Haft, Lafayette, Ind.
Albert S. Hibbs, Ada, O.
W. H. Heily, Columbus, O.
F. C. Higley, Delaware, O.
Chas. S. Hill, Chicago, Ill.
Jacob H. Hilkins, Indianapolis, Ind.
H. M. Hickok, Minneapolis, Minn.
G. S. Hird, Mitchell, Ind.
S. T. Henry, New York, N. Y.
Robt. A. Hoglund, Swea City, Ia.
John O. Hoglund, Swea City, Ia.

Arnold Hochstraner, Philadelphia, Pa.
Lewis H. Holladay, Chicago, Ill.
Norman G. Hough, Pittsburgh, Pa.
W. W. Horner, St. Louis, Mo.
James G. Houghton, Minneapolis, Minn.
M. A. Hoyt, Chicago, Ill.
J. E. Huber, Springfield, Ill.
O. C. Hubbard, Chicago, Ill.
H. J. Hurlbut, Chicago, Ill.
D. A. Hultgrew, Chicago, Ill.
Richard L. Humphrey, Philadelphia, Pa.
Jos. Husband, Chicago, Ill.
Jos. B. Houseworth, Philadelphia, Pa.
Harold D. Hynds, New York, N. Y.
R. Iben, Peoria, Ill.
S. H. Ingeberg, Chicago, Ill.
C. M. Ingram, Nashville, Tenn.
W. H. Insley, Indianapolis, Ind.
Fred. K. Irvine, Chicago, Ill.
Fred. K. Irvine, Chicago, Ill.
Nathan C. Johnson, New York, N. Y.
A. J. Johnson, Buffalo, N. Y.
S. H. Johnson, New York, N. Y.
T. H. Johnson, Sioux City, Ia.
J. Kahn, Youngstown, O.
C. V. Kerch, Jamesville, Ky.
H. D. Kerr, New York, N. Y.
L. J. N. Keliker, Indianapolis, Ind.
Harold E. Ketchum, Cleveland, O.
S. N. Kielley, Duluth, Minn.
Jas. F. King, Lake Forest, Ill.
Wm. M. Kinney, Chicago, Ill.
H. C. Koch, Kansas City, Mo.
H. F. Kopf, Chicago, Ill.
E. W. Kraft, Chicago, Ill.
W. H. Kremer, Chicago, Ill.
W. H. Kuhn, Chicago, Ill.
Paul R. Kirstein, Cincinnati, O.
Alfred E. Kornfeld, New York, N. Y.
Walter S. Lacher, Chicago, Ill.
T. F. Laist, Chicago, Ill.
G. N. Lamb, St. Charles, Ill.
Chas. F. Lang, Cleveland, O.
Louis J. Larson, Champaign, Ill.
J. W. Lathers, Beloit, Wis.
R. A. Lauth, Chattanooga, Tenn.
W. E. Law, Kingsport, Tenn.
Frank J. Lawson, Oxford, Ind.
R. W. Lesley, Philadelphia, Pa.
E. B. Lewis, St. Paul, Minn.
R. J. Lewis, Ft. Madison, Ia.
J. H. Libberton, Chicago, Ill.
Charles P. Light, Washington, D. C.
A. E. Lindon, Chicago, Ill.
J. Lineham, Chicago, Ill.
N. M. Loney, New York, N. Y.
John W. Lowell, Chicago, Ill.
Fred. W. Lumis, Springfield, Mass.
J. H. Lusby, Amherst, Nova Scotia.
Daniel B. Luten, Indianapolis, Ind.
Jos. McCarty, Kaukeuna, Wis.
D. C. McConaughy, Gary, Ind.
J. M. McCoy, Springfield, Ill.
Ernest McCullough, Chicago, Ill.
C. B. McCullough, Ames, Ia.
A. B. McDaniel, Chicago, Ill.
Cassius M. McDonald, Chicago, Ill.
M. F. McFarland, Ft. Madison, Ia.
W. A. McIntyre, Ardmore, Pa.
F. B. McKimnell, Topeka, Kan.
F. R. McMillan, Minneapolis, Minn.
Wm. J. McDermott, New York, N. Y.
A. B. McMillan, New York, N. Y.
D. H. McNally, Chicago, Ill.
C. W. Malcolm, Chicago, Ill.
B. M. Mathias, Minneapolis, Minn.
Arthur J. Maynard, State Farm, Mass.
Edward Meirs, Vandalia, Ill.
E. J. Mehren, New York, N. Y.
L. J. Mensch, Chicago, Ill.
Carl F. Meyer, Minneapolis, Minn.
R. T. Miller, Chicago, Ill.
Guy G. Mills, Chicago, Ill.
Geo. E. Martin, La Fayette, Ind.
E. J. Moore, New York, N. Y.
John W. Moore, New York, N. Y.
Robt. J. Moorehead, New York, N. Y.
F. A. Moorhead, West Jefferson, O.
H. H. Morgan, Chicago, Ill.
Albert Moyer, New York, N. Y.
John W. Mueller, New Castle, Ind.

J. W. Mullen, St. Paul, Minn.
Geo. W. Myers, Columbus, O.
L. H. Murphy, Chicago, Ill.
L. C. Mynhart, Ithaca, N. Y
C. E. Nagel, St. Paul, Minn.
W. Neale, Paw Paw, Mich.
J. Gale Needham, New York, N. Y.
W. J. Neidhart, Spencerville, Ind.
J. P. Newman, New York, N. Y.
Oscar Newhouse, Brandon, Miss.
A. T. North, Chicago, Ill.
Geo. A. Olsen, Chicago, Ill.
C. F. Ott, Chicago, Ill.
H. G. Overholt, Minneapolis, Minn.
Harvey Owen, St. Louis, Mo.
L. S. Packman, Chicago, Ill.
Ralph H. Pardee, Pontiac, Mich.
C. Cameron Parker, Toronto, Canada
A. W. Pearson, Chicago, Ill.
B. S. Pease, Chicago, Ill.
Mac W. Pederson, Sheridan, Ill.
Vernon M. Pierce, Washington, D. C.
Pract & Perrine, Pittsburgh, Pa.
C. C. La Pierre, Montreal, Canada.
Wm. H. Pickering, Spencerville, O.
H. E. Plumer, Buffalo, N. Y.
Harry F. Porter, Chicago, Ill.
Henry Pothast, Indianapolis, Ind.
W. G. Potter, Chicago, Ill.
N. S. Potter, Chelsea, Mich.
C. M. Powell, Chicago, Ill.
C. F. W. Price, Toronto, Canada.
John C. Pritchard, St. Louis, Mo.
Geo. A. Rankin, Washington, D. C.
B. H. Rader, Chicago, Ill.
Frank Raschig, Cincinnati, O.
H. S. Raymond, Waterloo, Ia.
A. M. Read, Columbus, O.
John L. Reeves, Chicago, Ill.
Geo. A. Ricker, Albany, N. Y.
M. L. Rice, Marion, Ind.
F. E. Rickart, Chicago, Ill.
R. L. Rickman, Eau Claire, Wis.
Robt. Ridgway, New York, N. Y.
David M. Riff, Chicago, Ill.
J. F. Rhodes, Montreal, Canada
V. D. L. Robinson, Chattanooga, Tenn.
A. E. Robinson, Chicago, Ill.
H. B. Robinson, East Chicago, Ind.
W. E. Rolfe, St. Louis, Mo.
F. L. Roman, Springfield, Ill.
O. H. D. Rohwer, Chicago, Ill.
H. R. La Roy, Chicago, Ill.
Wm. A. Royce, Travers City, Mich.
W. W. Rushmore, San Antonia, Tex.
Chas. E. Russell, Lake Forest, Ill.
H. J. Russell, Chicago, Ill.
J. S. Ryan, Alberta, Canada
J. J. Ryan, Muscatine, Ia.
Ald Jon. Ryan, Minneapolis, Minn.
Geo. J. Saffert, Fairfax, Minn.
C. R. Sandstrom, Chicago, Ill.
Frank N. Savage, Chicago, Ill.
Geo. Schmidt, Muscatine, Ia.
Fred W. Schultz, New York, N. Y.
Ralph W. Schainwald, Jr., New York, N. Y.
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